# OPTIMAL COST DESIGN OF RIGID RAFT FOUNDATION

*A Thesis Submitted*

in Partial Fulfilment of the Requirements

for the Degree of

M.TECH.

*by*

ANURADHA DAS

*to the*

**DEPARTMENT OF CIVIL ENGINEERING**

INDIAN INSTITUTE OF TECHNOLOGY KANPUR

March, 1998

# CERTIFICATE

It is certified that the work contained in the thesis entitled "**Optimal Cost Design of Rigid Raft Foundation**", by **Anuradha Das** has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

P. Basudhar

*Dr. P. K. Basudhar*

Professor

Dept. of Civil Engineering

I.I.T. Kanpur

March, 1998

ACKNOWLEDGEMENTS

First of all I would like to express my heartfelt gratitude and sincere thanks to Professor P.K.Basudhar for his valuable suggestions, sincere help and motivative encouragement throughout the all stages of my thesis work. It was a nice experience for me to work under his guidance.

I am greatful to all the teachers within and outside of my department who taught me during my M.Tech. programme.

I am thankful to my friend Sudip Paul for his help during my course work and thesis work.

I must mention the names of my friends Sujatha, Lakshman, Balamurali, Putta Raju, Arun and others who gave me nice company during my days in IIT Kanpur.

My heartfelt thanks to all the staff members of the geotechnical engineering laboratory who helped me during my laboratory work.

Anuradha Das.

# Contents

# List of Figures

# List of Tables

# Chapter 1

# INTRODUCTION

From engineering point of view any design should be safe and cost effective. In civil engineering, during designing a system, most of the times attention is only paid to safety considerations resulting in uneconomic design. On the other hand, to get a most economic i.e., optimum design many repeatative computations are to be carried out which are very much time consuming, tedious and monotonous. But the same can be achieved within a very short time with the help of a high speed digital computer. Sophisticated algorithms need to be used for the analysis to minimize the wastage of computer time. In the study reported here, the usefulness of such techniques in the optimal design of raft foundation has been presented.

For the economic design the objective function (cost function) is to be minimized subjected to some design restrictions arising out of material behavior and geometric considerations. As such, the problem is one of mathematical programming. Solution for such problems stretches the limits of conventional methods of calculus. Variational calculus is sometimes helpful but they are often limited in their range and versatility to tackle difficult problems.Many established optimization algorithms are available in literature. However, applications of these methods to new areas need

critical evaluation as no numerical method can be the panacea that can solve all problems and its efficiency is problem oriented.

With the advent of high speed digital computers these methods started finding their applications in different branches of engineering. This led to the development of various nongradient and gradient based algorithms for unconstrained minimization. Some of these methods are quite stable and efficient in finding the minimum. Subsequently many direct and indirect methods of minimizing a function have been developed to solve constrained problems. With these developments more and more avenues were opened to tackle difficult systems which could not be taken up earlier. Applications of these methods in Civil Engineering and specially in Structural Engineering began in the sixties. However, in Geotechnical Engineering their applications started only in the early seventies. There after for over two decades lot of these methods found their application in the area of slope stability analysis. However, their applications in the other areas of Geotechnical Engineering are very limited. So, there is a need to take up such studies. A brief review of the applications of these methods in geotechnical engineering is presented as follows.

Design of a raft foundation involves proportioning and structural design in which proportioning part mainly depends upon geotechnical aspects, i.e., bearing capacity and settlement of the soil. Adequate bearing capacity and restricted settlement, these two are the primary and most important criteria for design of foundation. An intergated analysis of raft foundation from geotechnical and structural engineering aspects is very essential, which is rarely done. So there is a scope for optimum cost design of raft foundation considering these two aspects in design.

Plenty of research work on the application of optimization methods

to be automated slope stability analysis are available in literature. Good reviews have been presented by various investigators (Bromhead, Groham, Nash, Choudhury, Bhattacharya) and, as such, those are not repeated here. Applications of these methods to other areas like optimal design of foundations (shallow or deep) and earth retaining structures are scanty. Some of these are as follows.

Subbarao et al (1972) have developed a method for an optimal dimensioning of footings subjected to axial loading and uniaxial moment. But this method does not take care of the tilt of footing. Bhavikatti et al (1979) have proposed a method of optimal design of isolated column footing using linear programming technique and reported about the progress of optimization and results of parametric studies. They concluded that using this method one can achieve 8 to 10 percent economy. But in this method settlement has not been considered. Madan Mohan (1984) has presented a methodology for settlement controlled optimum cost design of shallow footing. He worked on rectangular footing with generalized loading and used Sequential Unconstrained Minimization Technique. He has also reported a method of optimum plan dimensioning of a group of footings. Desai et al (1985) reported an optimum cost design of isolated footing and presented the results of different parametric studies. However, the study has considered only the axial loading on the footing. Mondal (1988) has studied the optimum cost design of shallow circular footings subjected to generalized loading. He has used Sequential Unconstrained Minimization Technique and reported about its success. Optimum design of a combined footing has been done by Pandian et al (1994). They have considered both bearing capacity criteria and settlement criteria to get the optimal solution for two columns. They restricted the settlement to 50 mm and

observed the changes of size of the footing with respect to the change of column loading. Only width of the footing has been considered as a design variable.

From the brief literature review presented above it can be concluded that till now no work has yet been done on the optimum design of raft foundation. For optimum cost design of such a foundation several trial designs need to be carried out satisfying all the structural and geotechnical design restriction. Even then enough cost reduction may not be achieved if the computations are done manually. All these can be easily achieved by resorting to optimization formulation. As such, in this thesis optimal cost design of rigid raft with several column loads positioned in a symmetric way has been reported.

Sequential Unconstrained Minimization Technique has been used in the presented study. Extended Penalty Function Technique (Kalvie and Moe, 1971) supplemented by Powell's Conjugate Direction algorithm (Powell, 1964) coupled with quadratic interpolation technique (Fox, 1971) has been used. The above approach has been found to be quite efficient by various investigators in tackling problems even when the initial design vector is infeasible. As the success of any numerical scheme is problem oriented, it is necessary to have an in-depth study and a critical appraisal of the same to arrive at any meaningful conclusion regarding their efficacy and effectability. Such a study has been reported in this thesis. Parametric studies have also been carried out to bring out the effect of these these on the total cost.

The organization of the thesis is as follows.

In Chapter 1, a brief introduction and literature review, the motivation and scope of the work have been briefly outlined.

Detailed analysis procedure identifying the objective function, the design constraints, and nonlinear programming formulation to isolate the optimal solution is presented in Chapter 2.

Results and discussions regarding the efficacy of the adopted technique and the influence of several design parameters on the overall cost have been presented in Chapter 3. Finally in Chapter 3, the scope of future study has been reported.

# Chapter 2

# GENERALIZED PROCEDURE FOR OPTIMAL COST DESIGN OF RAFT FOUNDATION

## 2.1 GENERAL

Design of a rigid raft foundation involves, design of a slab, having a very high thickness. In the conventional approach of designing, a rigid raft can be divided into a number of strips in both longitudinal and transverse directions. These strips are treated as combined footings subjected to several column loads. Structural design of the same is carried out using IS code of practice.

Raft foundations can be adopted for steel as well as concrete super-structures. For concrete super-structure, columns can be supported directly by the raft and no pedestal is required. But, for the steel structures pedestals are provided to connect steel super-structures with the concrete foundations. For maintenance purpose the joint should be placed at a higher level than ground surface. In this study for generalized column loading conditions both the options have been kept open for generalized column loading conditions. Limit State design method has been adopted

for concrete design.

## 2.2 STATEMENT OF THE PROBLEM

The Fig. 2.1 shows a rectangular raft with concrete columns while Fig. 2.2 shows the same with steel columns. The connection of the raft with the super-structure may be either made of concrete or steel as shown in the figures. The problem is to find the optimal cost design of the raft for a given site condition, position of columns and the loading conditions (axial loading and biaxial moments as shown in Fig. 2.3).The objective is to find the optimal cost design under the following two considered cases :

1. Design of raft foundation for a set of concrete columns of given dimensions.

2. Design of pedestals and raft foundation for a set of steel columns of given dimensions.

## 2.3 ANALYSIS

### 2.3.1 Assumptions

In developing the generalized procedure of analysis the following assumptions are made regarding the shape of the raft, columns, pedestals, spacing of the columns and the nature of the soil medium.

1. Shape of the raft, columns and pedestals are rectangular or square.

2. Spacings of the columns are same along x-direction.

3. Spacings of the columns are same along y-direction.

4. Soil is linearly elastic.

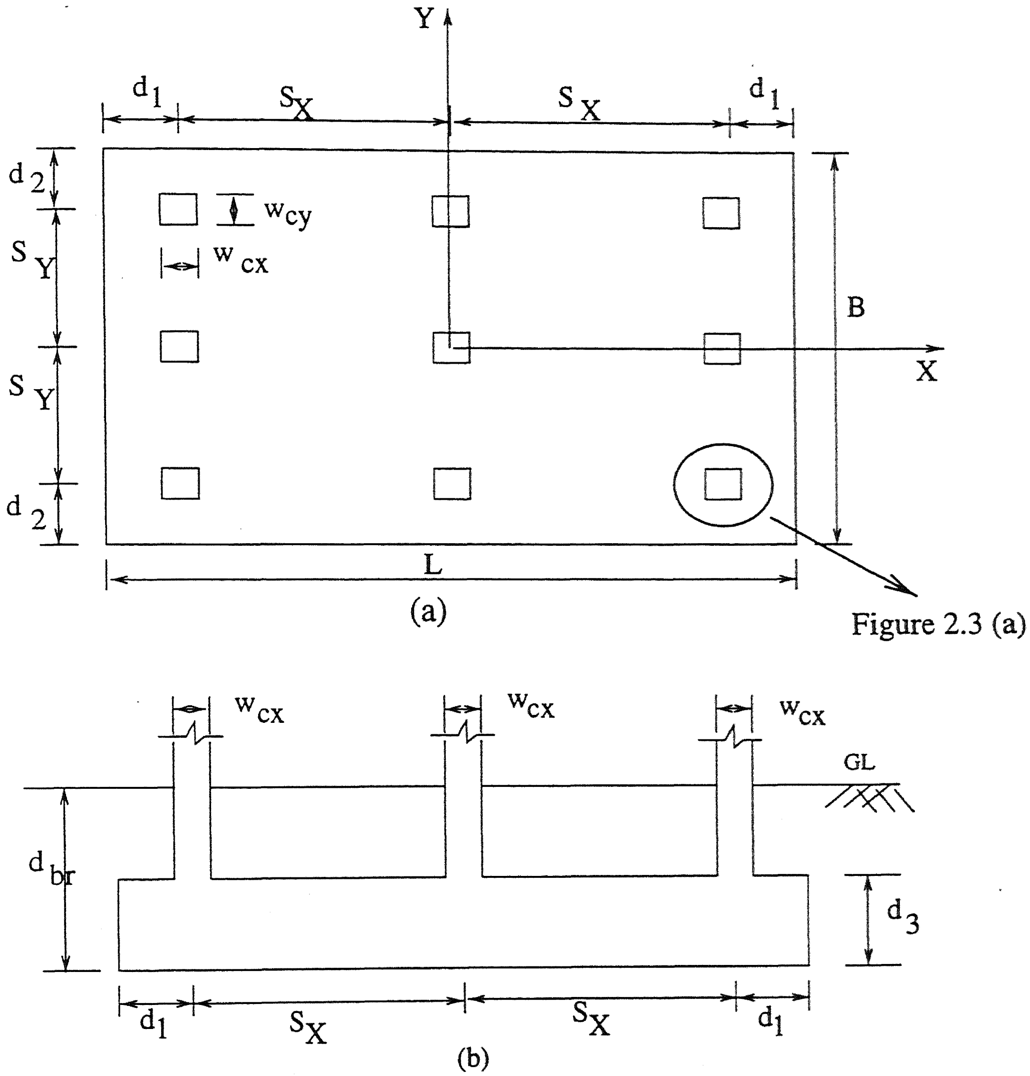



Figure 2.1: Raft for Concrete super-structure, (a) plan, (b) elevation

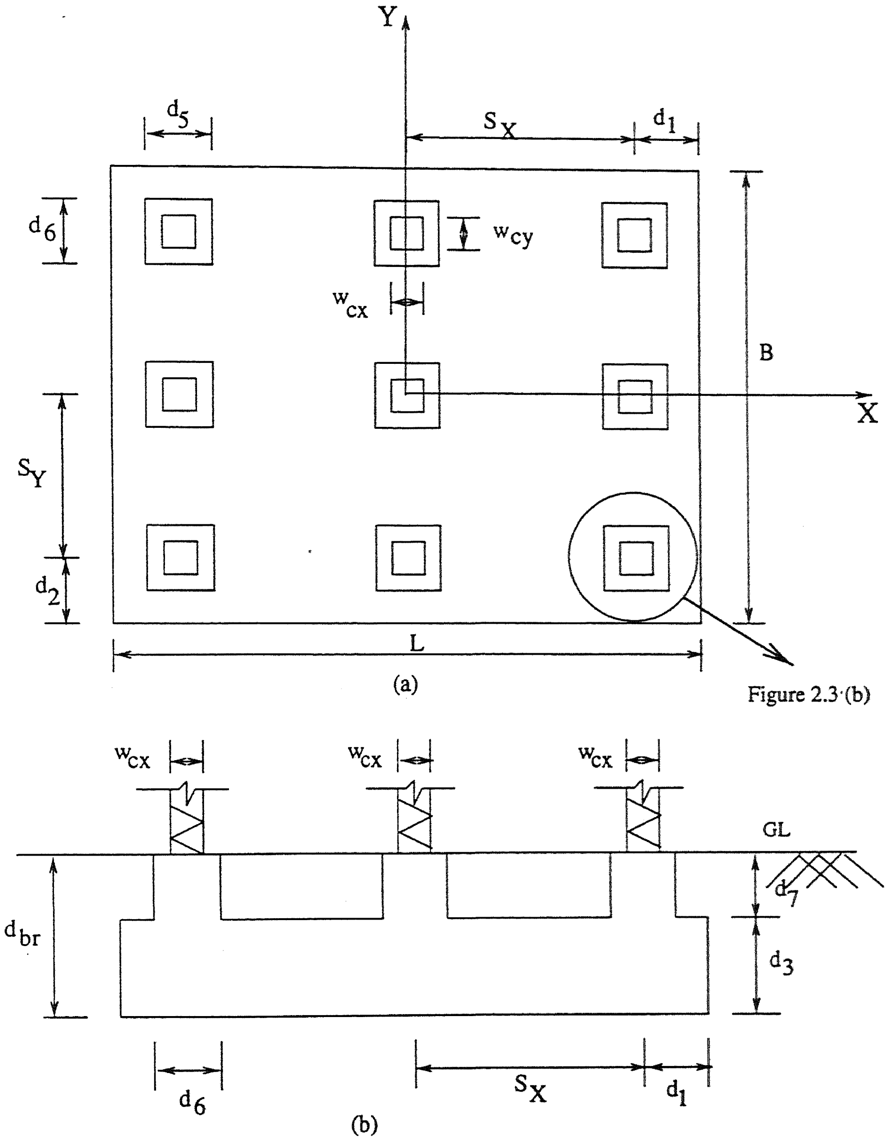
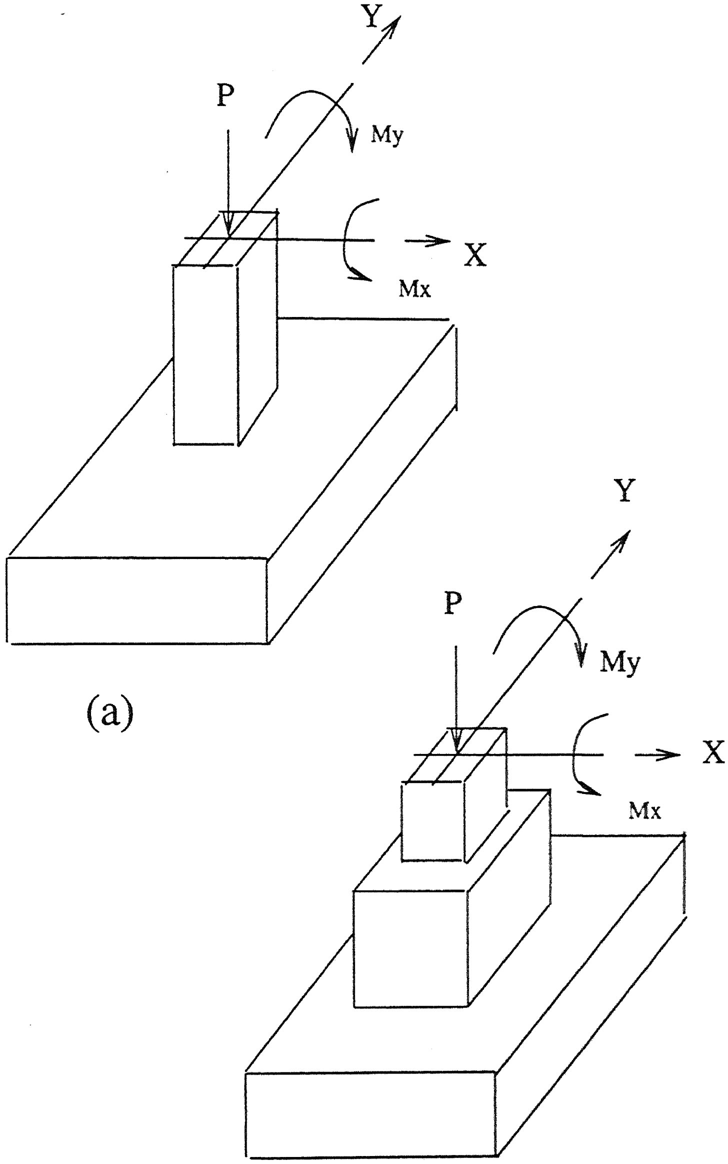



Figure 2.2: Raft for Steel super-structure, (a) plan, (b) elevation




(a)

(b)

Figure 2.3: Enlarge view of loading, (b) steel column, (a) concrete column

### 2.3.2 Design Variables

When the raft has to be designed for concrete columns, number of design variables are 4. They are as follows :

1. edge distance in x-direction, $d_1$
2. edge distance in y-direction, $d_2$
3. thickness of the raft, $d_3$
4. diameter of the bar, $d_4$

   When the raft has to be designed for steel columns, with the above variables three more variables, i.e., the dimensions of the pedestals are also to be included. They are as follows :

5. width of the pedestal in x-direction, $d_5$
6. width of the pedestal in y-direction, $d_6$
7. height of the pedestal, $d_7$.

### 2.3.3 Objective Function

Total cost of the raft is considered as the objective function of the problem. It includes the cost of excavation, cost of construction, cost of concrete, cost of steel and cost of filling. If the raft is for steel super-structure cost of pedestal is to be included with the total cost of the raft.

Objective function is given by,

$$\begin{aligned} F &= C_{exc} + C_{conc} + C_{steel} + C_{fill} \\ &= R_{exc}\, V_{exc} + R_{conc}\, V_{conc} + R_{steel}\, V_{steel} + R_{fill}\, V_{fill} \end{aligned}$$

where, $C_{exc}$, $C_{conc}$, $C_{steel}$ and $C_{fill}$ are the total costs of excavation, concrete (including construction charges), steel (including construction charges), and filling respectively. $R_{exc}$, $R_{conc}$, $R_{steel}$ and $R_{fill}$ are rates of the items and $V_{exc}$, $V_{conc}$, $V_{steel}$, $V_{fill}$ are the volumes of excavation, concrete, steel and filling respectively.

Total volume of excavation $V_{exc}$ is calculated in the following way,

$$V_{exc} = L\ B\ d_{br}$$

where,

$L$ and $B$ are the length and width of the raft in $m$

$d_{br}$ = depth of raft base from the ground level in $m$

$$L = 2\ d_1 + (n_{cp} - 1)\ S_x$$

$$B = 2\ d_2 + (n_{rp} - 1)\ S_y$$

where, $d_1$ and $d_2$ are the design variables as described earlier

$n_{cp}$ and $n_{rp}$ are the numbers of columns in each row parallel to x-direction and y-direction respectively.

$S_x$ and $S_y$ are the spacing in $m$ between two columns along x-direction and y-direction respectively

Total volume of the concrete is calculated in the following way

$$V_{conc} = L\ B\ d_3 - V_{steel}$$

where,

$d_3$ = thickness of the raft in $m$

other things are mentioned above.

In case of steel super-structure with the above volume of concrete $V_{conc}$,

the volume of concrete required for the pedestals is to be added. Concrete volume for pedestals is given by,

$$V_{ped} = n_{cp}\, n_{rp}\, d_5\, d_6\, d_7$$

As the volume of steel required for the pedestals is very less compared to the volume of concrete, volume of steel is neglected during the calculation of volume of concrete.

Volume of steel is calculated as follows. At the points of maximum bending moment occurring at the face of columns and in the span (the length between two columns) required reinforcement is calculated. The total volume of steel is given by,

$$V_{ast} = \sum_{i=1}^{n} P_{sti}$$

where,

$n$ = number of points where reinforcement is calculated

$P_{st}$ = reinforcement required at each point of calculation $= N\, a\, l$

$N$ = numbers of bar required

$a$ = area of cross-section of a single bar

$l$ = length of each bar taking the consideration of development length and anchorage length.

As the thickness of a raft is very high, reinforcement is to be provided at both top and bottom faces. Method of reinforcement calculation is given later.

Volume of filling $V_{fill}$ is calculated in the following way. For concrete super-structure it is,

$$V_{fill} = V_{exc} - V_{conc} - V_{ast} - V_{col}$$

where, $V_{col}$ =total volume of columns under the ground level up to top of the raft

For steel super-structure it is,

$$V_{fill} = V_{exc} - V_{conc} - V_{ast} - V_{ped}$$

where, $V_{ped}$ = Total volume of pedestals under the ground level.

### 2.3.4 Design Constraints

For the design to be safe several design restrictions from the structural and geotechnical considerations are imposed. These are discussed under separate headings as followes.

A. *Structural Design Constraints*

1. The value of design variables $d_1$ and $d_2$ should be such that it will be at least equal to half of the column dimension in that direction.Mathematically,

$d_1 \geq w_{cx}/2$ and $d_2 \geq w_{cy}/2$

in case of concrete super-structure.

$d_1 \geq d_5/2$ and $d_2 \geq d_6/2$

in case of steel super-structure.

where, $w_{cx}$ and $w_{cy}$ are column dimensions in x-direction and y-direction respectively.

$d_5$ and $d_6$ are pedestal dimension in x-direction and y-direction respectively.

2. If the dimensions of the design variables $d_1$ and $d_2$ are very large, then due to soil pressure high bending moment will develop at the outer face of column and it may cause failure of the structure. So the maximum dimensions of $d_1$ and $d_2$ are restricted to 1 metre. Mathematically

$d_1 \leq 1m$ and $d_2 \leq 1m$

3. In case of rigid raft thickness of the raft is very high. But it should not be so thick requiring excessive amount of concrete and steel and produce an uneconomic design. Again, it should not be so thin that the raft cannot be considered as a rigid one. So thickness of the raft is restricted as followes (Bowels, 1886) :

$d_3 \geq 0.75m$ and $d_3 \leq 2.0m$

4. Diameter of the bar is also kept as a design variable. The size of the bar is restricted to :

$d_4 \geq 12mm$ and $d_4 \leq d_3/8$

In case of steel super-structure in addition to all the constraints already specified above the following constraints are applicable.

5. Plan dimension of the pedestal at least 0.5m greater than the width of the column in the considered direction. Mathematically,

$d_5 \geq w_{cx} + 0.5$ and $d_6 \geq w_{cy} + 0.5$

6. Maximum width of the pedestal is restricted to,

$d_5 \leq 1.5m$ and $d_6 \leq 1.5m$

7. Height of the pedestal is restricted to

$d_7 \geq 0.2m$ and $d_7 \leq 2.0m$

8. Height of the pedestal is restricted by the following manner also

$d_{br} \geq d_3 + d_7$

9. To check if the provided depth of the pedestal is sufficient or not,

for each pedestal the following relation should be satisfied [IS:456, clause 33.1.3] :

$$\tan\alpha \geq 0.9\sqrt{\frac{100\,q_o}{f_{ck}} + 1}$$

where,

$q_o$ = calculated maximum bearing pressure at the base of the pedestal in $N/mm^2$

$f_{ck}$ = characteristic strength of concrete in $N/mm^2$

Beside these structural constraints there are some other constraints applicable for detailing part of the reinforcement. As per IS:456 those constraints are given below.

(a) Requirement of minimum steel [IS:456, Clause 25.5.1.1 (a)] :

$$A_{smin} = \frac{0.85}{f_y}$$

where, $f_y$ = characteristic strength of steel in $N/mm^2$

(b) Requirement of maximum steel [IS:456, Clause 25.5.1.1 (b)] :

$$A_{smax} = 0.04\, b\, D$$

where,

$b$ = width of the section

$D$ = overall depth of the section

Provided steel must be in between the above limits, i.e.,

$$A_{smin} \leq A_{st} \leq A_{smax}$$

(c) Requirement of minimum spacing :

$$S_{min} = d_4$$

where, $S_{min}$ = minimum spacing in between two bars

$d_4$ = maximum diameter of bar used for design

(d) Requirement of maximum spacing :

$S_{max} = 450mm$ and $S_{max} = 3\ d_3$

whichever is less.

Provided spacing must be within the maximum and minimum values as specified above [IS:456, Clause 25.3.1], i.e.,

$$S_{min} \leq S_{st} \leq S_{max}$$

. $S_{st}$ = spacing of the reinforcing steel.

Reinforcement length should be such that it is in excess of the estimated length by a length called development length ($L_d$) estimated by the following relation [IS:456, Clause 25.2.1].

$$L_d = \frac{d_4 \sigma_s}{4 \tau_{bd}}$$

where, $d_4$ = diameter of the bar

$\sigma_s$ = stress in bar at the section considered at design load

$\tau_{bd}$ = design bond stress.

For the members, subjected to combined axial load and biaxial bending, following test should be done [IS:456, Clause 38.6] :

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1$$

where,

$M_{ux}$ and $M_{uy}$ are moments about x and y direction due to design loads.

$M_{ux1}$ and $M_{uy1}$ are maximum uniaxial moment capacity for axial load $P_u$, bending about x and y directions respectively. $\alpha_n$ is related to $\frac{P_u}{P_{uz}}$.
$P_{uz}$ is given by :

$$P_{uz} = 0.45 f_{ck}\, A_c + 0.75 f_y\, A_{sc}$$

where, $P_u$ = axial load on the column
$A_c$ = area of concrete
$A_{sc}$ = area of steel.

B. *Geotechnical Constraints*

Constraints imposed on the design from geotechnical considerations are :

1. Settlement of the raft should be within permissible limits
2. Imposed stresses should be less than the allowable bearing capacity.

These constraints are described below.

Settlements

Main aspect of foundation design is to restrict the total and differential settlements. Due to applied load on the raft, pressure on the soil will increase and it results some settlement of the underneath soil. Method used for calculation of stresses and settlement is as follows. For estimating immediate settlement Boussinesq's equation has been used. In case of cohesive soil there will be both immediate settlement and consolidation settlement and by adding up these two, total settlement is obtained. Consolidation settlement is estimated by using Skempton Bjerrum (1957) method. However, for frictional soils the settlement may be computed either by using the Boussinesq's equation or by using Schmertmann method. The methods which are used are described below.

Estimation of Stresses and Displacements

For calculating settlement estimation of stress increment at different points within the soil mass is necessary. As the raft is considered as a rigid one, stresses are calculated in the following way. Applied load on the raft may or may not be symmetric. So, with respect to the centroid of total area, eccentricity of the loading is to be determined.

$$e_x = \frac{\sum P_i\, x_i}{\sum P_i} \tag{2.1}$$

$$e_y = \frac{\sum P_i\, y_i}{\sum P_i} \tag{2.2}$$

where,

$e_x$ and $e_y$ are the eccentricities of the loading along x-direction and y-direction respectively.

$P_i$ = concentrated column loads

$x_i$ and $y_i$ are the distances of $P_i$ from the centroid along x-direction and y-direction respectively.

With these eccentricity at several points under the raft stresses are calculated using the following equation.

$$p_{1,2} = \frac{\sum P_i + R}{A} \pm \frac{M_x + \sum P_i e_y}{I_x} y \pm \frac{M_y + \sum P_i e_x}{I_y} x \tag{2.3}$$

where, $M_x$ and $M_y$ are total concentrated column moments about x-direction and y-direction respectively.

$A$ = total area of the raft

$I_x$ and $I_y$ are moments of inertia of the raft about x-direction and y-direction respectively.

$x$ and $y$ are the distances of the point from c.g. where stress is to be calculated along x-direction and y-direction respectively.

$R$ = self weight of raft = $L\ B\ W_{con}$

$W_{con}$ = unit weight of concrete = $25kN/m^3$

The whole raft is divided into several small rectangular elements. For each rectangular element stress is calculated. As the stress distribution under the raft is considered as linear, calculation of stress at four corners of each element is easy. From the stresses at four corners average stress at the centre of the element is calculated. If $p_a$ is the average stress at the centre of the element,

$$P_a = p_a\ a_e$$

is considered as the concentrated load at the centre of the element, where, $a_e$ is the area of a single element. At any depth $z$, stress increment is calculated using the Boussinesq's equation given below.

$$\sigma_{zi} = \frac{3\ P_a\ z^3}{2\ \pi\ r^5} \tag{2.4}$$

where,

$\sigma_{zi}$ = stress at $z$ depth

$P_a$ = point load applied at the ground surface

Other quantities are as shown in the Fig. 2.4.

At the four corners of the raft elastic settlement is calculated using Eq. 2.5. Due a concentrated load $P_a$ acting at the surface of the soil settlement at any cylindrical radius $r$ and at any depth $z$, the settlement is given by the following equation.

$$\rho_z = P_a \frac{1+\nu}{2\pi E_s r}\left[2(1-\mu) + \frac{z^2}{r^2}\right] \tag{2.5}$$

where,

$\rho_z$ = vertical settlement of the soil

$E_s$ = modulus of elasticity of the soil

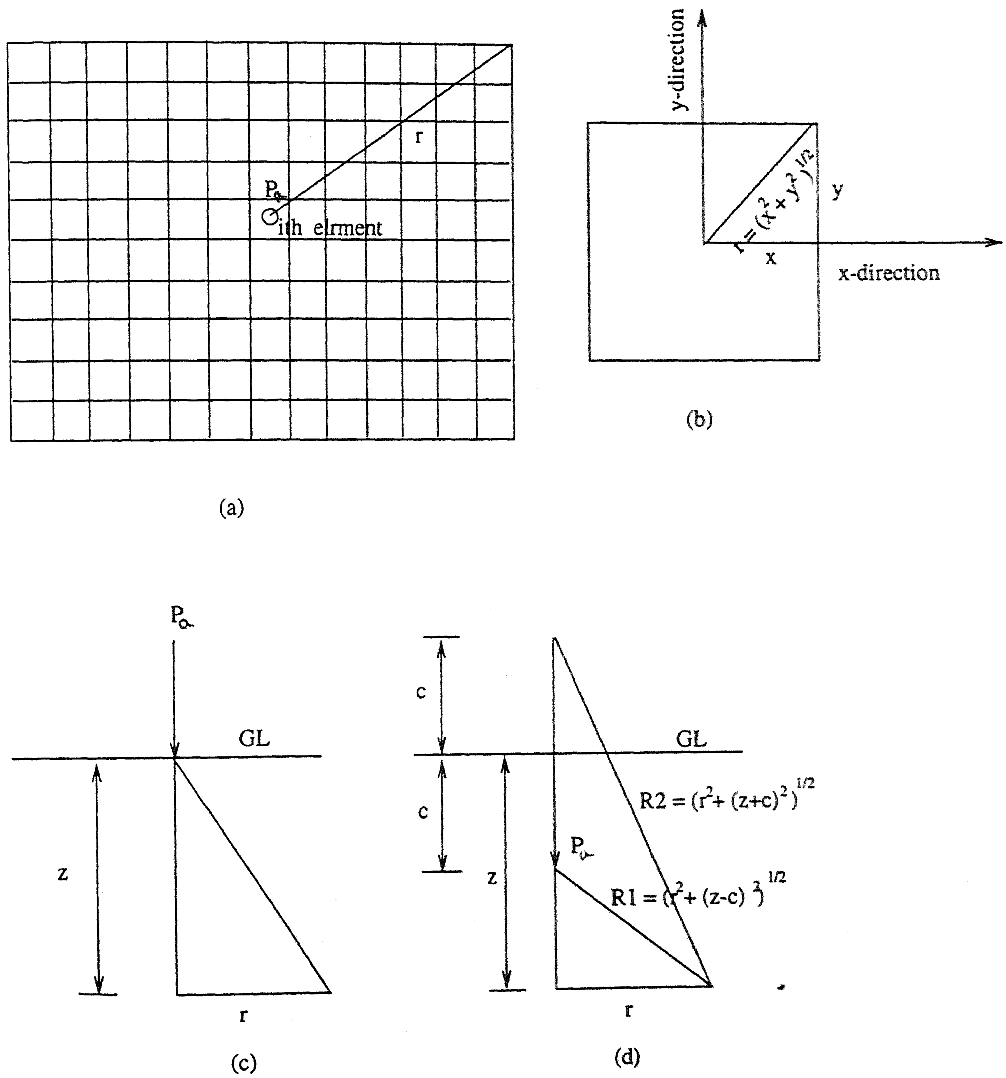



Figure 2.4: (a) Discretization of the raft, (b) Coordinate axes, Definition figure (c) Boussinesq and (d) Mindlin solution

$\mu =$ poison's ratio of the soil

Here an option is kept to check the result using Mindlin's approach. The equation is given below. Diagram is given in Fig. 2.4.

$$\rho_z = \frac{P_a}{16\,\pi\,G\,(1-\mu)}\left[\frac{3-4\mu}{R_1} + \frac{8(1-\mu)^2-(3-4\mu)}{R_2} + \frac{(z-c)^2}{R_1^3} + \frac{(3-4\mu)(z+c)^2-2cz}{R_2^3} + \frac{6cz(z+c)^2}{R_2^5}\right]$$

$P_a$ for each element will contribute to the settlement of each corner of the raft.

For the $ith$ element $P_{ai}$ causes $\rho_{zi}$ settlement then, total settlement at each corner can be stated as

$$\rho_z = \sum \rho_{zi}$$

With the same way settlement at the rest three corners can be calculated.

Consolidation Settlement

If the raft has to be placed on fine grained soils, then there will be immediate settlement as well as consolidation settlement. Immediate or elastic settlement can be calculated either by using Boussinesq's or Mindlin's settlement equation (Poulos and Davis, 1974). Then consolidation settlement can be calculated using the following formula :

$$S_c = \frac{C_c}{1+e_o}\,H\,\log\frac{\sigma_o' + \Delta p}{\sigma_o'} \tag{2.6}$$

where,

$C_c =$ compression index

$e_o =$ initial void ratio of the soil

$H =$ thickness of the soil

$\sigma_o^{'}$ = effective stress at the middle of the soil layer

$\Delta p$ = stress increment due to applied load using Boussinesq equation

If there are several layers, then total consolidation settlement can be calculated as :

$$S_c = \sum S_{cj}$$

where,

$S_{cj}$ = settlement of the $j^{th}$ layer.

$j$ is the number of layers.

Skempton-Bjerrum correction factor is applied on the result obtained from Terzaghi's one-dimensional consolidation formula to take care of the three-dimensional consolidation effect.

$$S_{actual} = \beta \, S_c$$

where, $\beta$ = Skempton-Bjerrum correction factor

The constant $\beta$ is a function of dimension of the footing (breadth, B), depth of the compressibe layer (z) and porepressure coefficient 'A'. The variation of $\beta$ with 'A' for various values of $z/B$ and for circular/strip footings are produced in graphical form.

As per IS:1904-1978, maximum allowable settlement for the raft placed on either clay or sand is $100mm$ and $50mm$ respectively. Differential settlement is calculated by estimating the difference of total settlement at two extreme corners. If $\rho_1$ is the settlement at corner 1 and $\rho_2$ is the settlement at corner two and $l$ is the distance between those points then tilt $(\theta)$ is given by

$$\theta = \frac{\rho_1 - \rho_2}{l}$$

Maximum tilt is restricted to 0.003, i.e.,

$$\theta \leq 0.003$$

•

Schmertmann Method

If CPT data is available for the site then this method can be used to determine total settlement. In this method the strain is assumed to increase from a minimum at the base of to a maximum of 0.6 at $B/2$ depth, then decrease and reaches zero at the depth equal to $2B$. The area of the strain triangle gives settlement. The triangular vertical strain influence factor distribution diagram as proposed by Schmertmann (1978) is shown in Fig. 2.5.

The equation for calculating settlement is :

$$S = C_1\, C_2\, q_n \sum_{0}^{2B} \frac{I_z}{E_s} \,\Delta\, z \tag{2.7}$$

where,

$S$ = total settlement

$q_n$ = net foundation base pressure = $(q - q_o^{'})$

$q$ = total foundation pressure

$q_o^{'}$ = effective overburden pressure at foundation level

$\Delta\, z$ = thickness of the elemental layer

$I_z$ = vertical strain influence factor

$C_1$ = depth correction factor

$C_2$ = creep factor

The equations for $C_1$ and $C_2$ are :

$$C_1 = 1 - 0.5\, (\frac{q_o^{'}}{q_n})$$

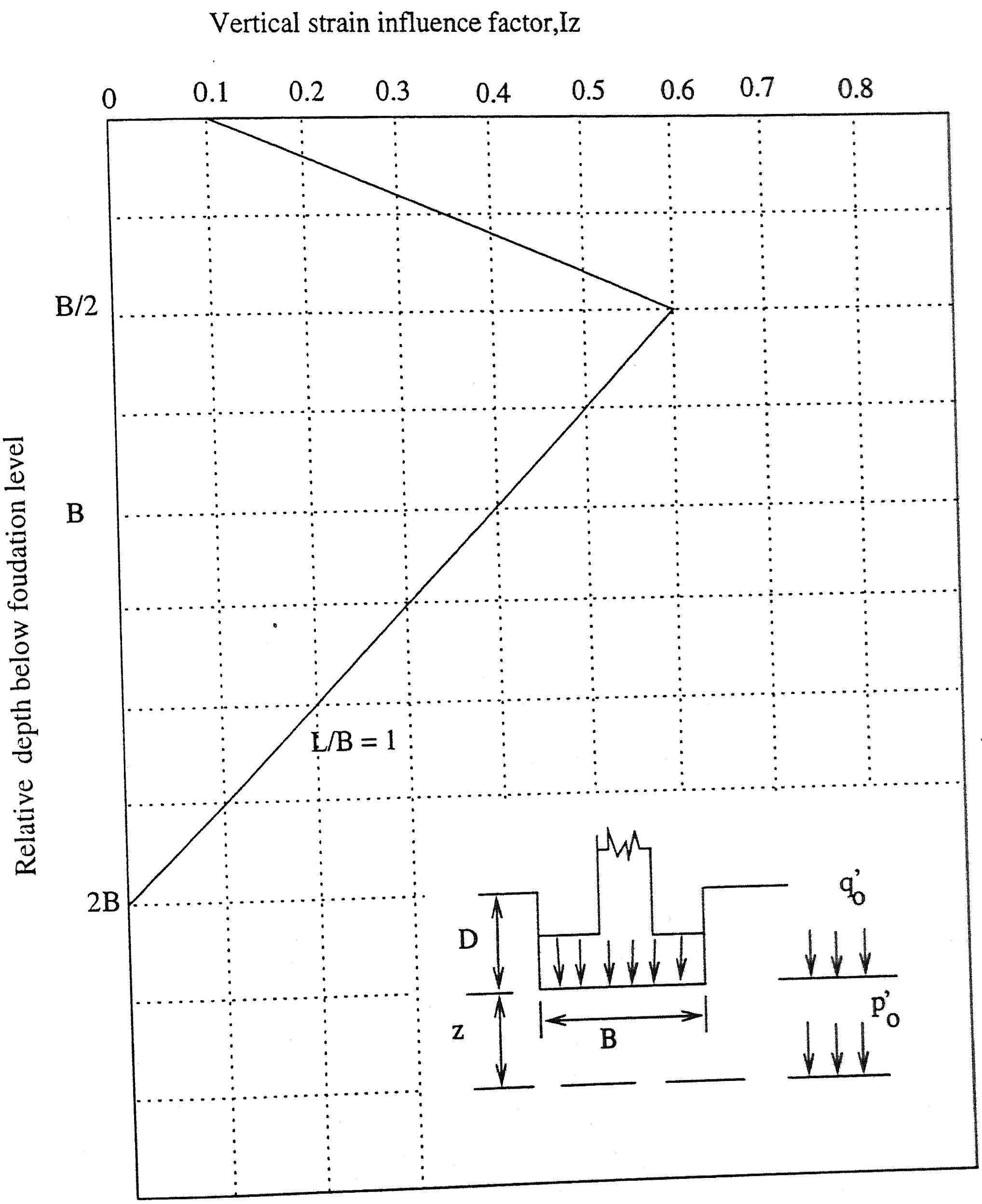



Figure 2.5: Vertical strain influence diagram

$$C_2 = 1 + 0.2\, log_{10}(\frac{t}{0.1})$$

where,

$t$ =time in years.

$E_s$ can be calculated from the following equation :

$$E_s = \alpha\, q_c \tag{2.8}$$

$\alpha$ depends upon the type of the soil.

$q_c$ = cone resistance A computer program has been developed to estimate the factor $I_z$ corresponding to any depth ($\leq 2B$).

## Bearing Capacity

This is the other important criteria for the design of a foundation. If the soil pressure exceed allowable bearing capacity of the soil then there will be shear failure of soil underneath the raft. Bearing capacity of the soil is calculated by using Hansen's ultimate bearing capacity equations (Bowels, 1982, 1988).

For $c - \phi - soil$ :

$$q_{ult} = c\, N_c\, s_c\, d_c\, i_c + q\, N_q\, s_q\, d_q\, i_q + \frac{1}{2}\, B\, \gamma\, N_\gamma\, s_\gamma\, d_\gamma i_\gamma \tag{2.9}$$

For $\phi - soil$ :

$$q_{ult} = q\, N_q\, s_q\, d_q\, i_q + \frac{1}{2}\, B\, \gamma\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma \tag{2.10}$$

For $c - soil$ only :

$$q_{ult} = 5.14\, Su\, (1 + s_c^{'} + d_c^{'}) + q \tag{2.11}$$

where, bearing capacity factors

$$N_q = e^{\pi\, \tan\phi} \tan^2(45 + \phi/2)$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = 1.5(N_q - 1)\tan\phi$$

shape factors

$$s_c' = 0.2\, B/L$$

$$s_c = 1 + N_q\, B/L$$

$$s_q = 1 + (B/L)\tan\phi$$

$$s_\gamma = 1 - 0.4\, B/L$$

depth factors

$$d_c' = \begin{cases} 0.4\, D/B & \textbf{if } D \leq B \\ 0.4\ \tan^{-1} D/B & \textbf{if } D > B \end{cases}$$

$$d_c = \begin{cases} 1 + 0.4\, D/B & \textbf{if } D \leq B \\ 1 + 0.4\ \tan^{-1} D/B & \textbf{if } D > B \end{cases}$$

$$d_q = \begin{cases} 1 + 2\ \tan\phi(1 - \sin)^2\ D/B & \textbf{if } D \leq B \\ 1 + 2\ \tan\phi(1 - \sin)^2\ \tan^{-1} D/B & \textbf{if } D \leq B \end{cases}$$

for all $\phi$

$$d_\gamma = 1.0$$

As all the loading conditions are considered as vertical. Inclination factors are taken as 1, i.e.,

$$i_c = i_q = i_\gamma = 1.0$$

. $S_u$ = undrained shear strength

$c$ = cohesion intercept

$\phi$ = angle of shearing resistance

$D$ = depth of the foundation

$B$ = width of the foundation

$q = \gamma\, D$

In Hansen's equations other factors like, ground factor and base factors are also there. But here, ground and base of the raft are considered as horizontal and as such those factors are not considered.

Factor of safety for raft against bearing capacity failure varies from 1.7 to 2.5 (Bowels, 1982, 1988).

So, allowable bearing capacity is calculated by the following way,

$$q_{all} = q_{ult}/F.S.$$

Here the adopted value of factor of safety is 2.0.

In ultimate bearing capacity equations for $\phi$ and $c - \phi$ soil there is a term including the dimension of the raft (width of the raft B). It seems that if B increases indefinitely then ultimate bearing capacity will also increase indefinitely to an infinite extent. But actually it does not occur in that way. Up to a certain size of foundation $q_{ult}$ increases and then it becomes constant though size of the foundation increases. Bowels (Bowels, 1988) has suggested a reduction factor $r_\gamma$ which can be calculated using the following formula if $B > 2m$ :

$$r_\gamma = 1 - 0.25\, log(B/K)$$

$K = 2.0$ for SI units.

This reduction factor is used in the following way,

$0.5\,\gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, r_\gamma$

This takes care of the effect of the size of the raft beyond a certain limit.

Now the bearing capacity constraints can be stated as :

If $q_r$ is the produced stress at the four corner of the raft then $q_r \leq q_{all}$

Check for Relative Rigidity of raft

It is well known that for design of beams on elastic foundation if the maximum spacing between two columns ($S$) is such so that the product of $\lambda$ and $S$ is less than $\pi/4$ then the beam is called as rigid one; $\lambda$ is a factor having the dimension of $1/length$ and can be calculated using the following formula :

$$\lambda = \sqrt[4]{\frac{K_s\, B_S}{4\, E_b\, I_b}} \tag{2.12}$$

where,

$K_s$ = modulus of subgrade reaction of the soil

$B_S$= width of the strip

$E_b$ = elasticity modulus of the material of the strip (concrete)

$= 5700\,\sqrt{f_{ck}}$

$I_b$ = moment of inertia of the section of the strip $= 1/12\, B\, d^3$

$d$ = depth of the beam

Modulus of subgrade reaction $K_s$ can be calculated using the following equation (Vesic, 1961a, 1961b) :

$$K_s = 0.65\,\sqrt[12]{\frac{E_s\, B_S^4}{E_b\, I_b}}\,\frac{E_s}{1-\mu^2} \tag{2.13}$$

where,

$E_s$ = elasticity modulus of the soil

$\mu$ = poisons ratio of the soil

For all the strips dimension of the width should be less such that the value of $\lambda$ should be sufficiently less so that the product of maximum spacing of the column in the raft and $\lambda$ will be less than $\pi/4$. In that case only the adopted method is applicable and as such, this check is very essential.

## 2.4 DESIGN

Once the different dimensions of the raft, depth of the foundation, loadings and soil parameters are known the structural design can be done. The method adopted here to design a raft is the conventional strip method. In this method the whole raft is taken as a combination of several strips according to the number of column rows in both x and y directions. Then each strip is designed separately. For design the following steps are followed.

1. All the vertical column loads and concentrated column moments coming on the whole raft about y-direction is taken into account. The maximum and minimum pressure at the two ends of the raft along x-direction is estimated by using the following equation.

$$px_{1,2} = \frac{\sum P_i + R_s}{a_S} \pm \frac{M_y + \sum P_i e_x}{I_y} x \tag{2.14}$$

where $R_s$ = total weight of the strip

$i$ = number of columns on that strip

2. The raft is divided into several strips, according to the number of column rows, parallel to the x-direction.

3. A single strip is considered parallel to x-direction. Eccentricity of the loading is calculated using Eq. 2.1. For this eccentricity and the concentrated moment on that strip maximum and minimum pressure is calculated at the two ends of the strip using Eq. 2.14.

Where, $i$ = number of columns on that particular strip

$R = R_s$ = self weight of the strip.

The stresses are $ps_{1,2}$. In Fig. 2.6 calculation of $px_{1,2}$ and $ps_{1,2}$ are shown.

4. The average of $px_{1,2}$ and $ps_{1,2}$ is calculated and the average stress is $pa_{1,2}$.

5. From the average stresses $pa_{1,2}$ total soil pressure $S_P$ is calculated by the following way :

$$S_P = 1/2\ (pa_1 + pa_2)\ L_s\ B_s$$

where, $L_s$ = length of the strip

$B_s$ = width of the strip

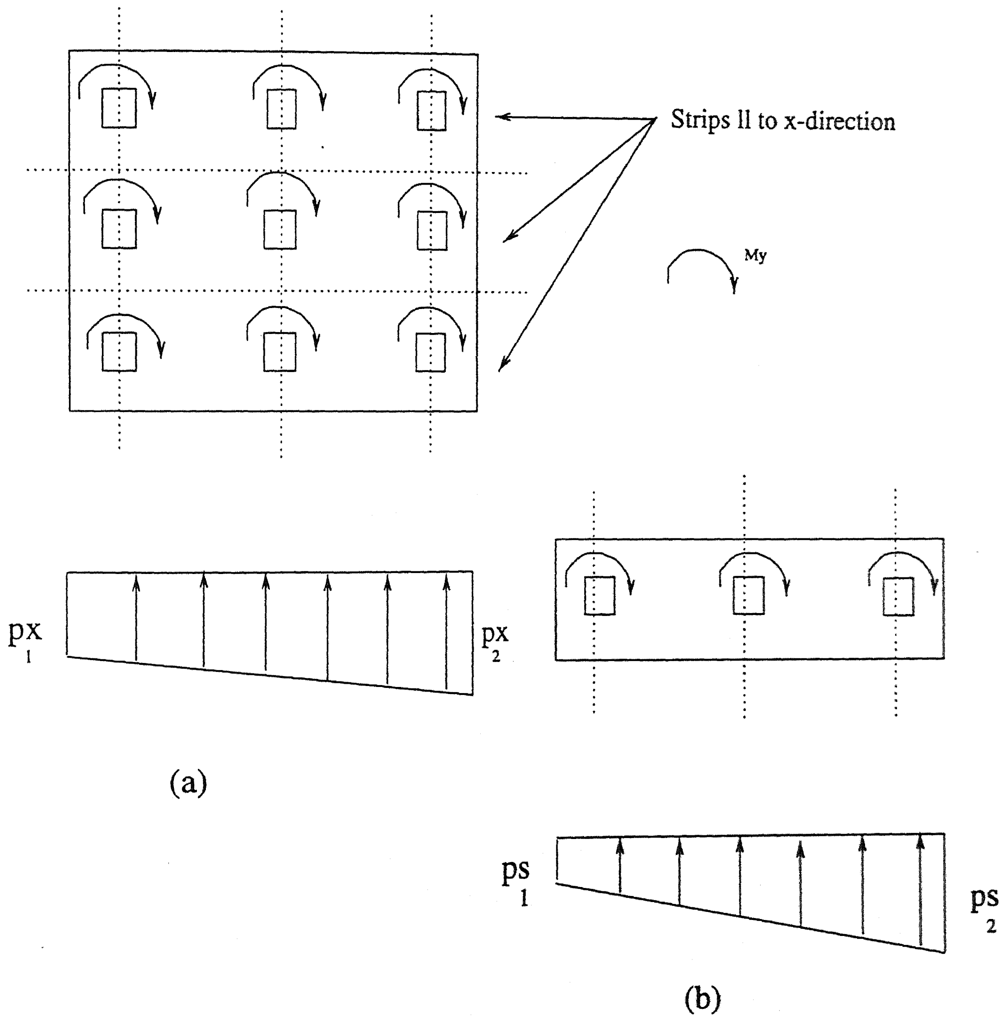



Figure 2.6: Stress calculation for design, (a) Considering whole raft, (b) Considering one strip

6. If the total load on the strip is $L_T$ including the self weight of the

raft then the load reduction factor $L_{RF}$ can be calculated in the following way.

$$L_{RF} = S_P/L_T$$

. All the concentrated column loads and the self weight of the strip is multiplied by $L_{RF}$ and the obtained loads are used to calculate design moments.

This has been done to satisfy force equilibrium condition for that particular strip (Bowels, 1982). The same thing has been done for moments for that particular strip. If the moment reduction factor is given by $M_{RF}$, then all the concentrated moments are multiplied by $M_{RF}$ and the obtained moments are used subsequently in the design moment calculation. At each face of the column and at the span in between two columns bending moments are calculated. At the face of the column maximum bending moment is considered as design bending moment at that column point and maximum span moment is considered as design span moment at that span. The following equation given in Appendix E of IS:456 is used.

the required reinforcement at those points.

$$M_u = 0.87\ f_y\ A_{st}\ d\ (1 - \frac{A_{st}\ f_y}{b\ d\ f_{ck}}) \tag{2.15}$$

where,

$M_u$ = moment for which reinforcement has to be provided

$f_y$ = characteristic strength of steel in $N/mm^2$

$f_{ck}$ = characteristic strength of concrete in $N/mm^2$

$d$ = effective depth of the section

$b$ = width of the section

$A_{st}$ = area of reinforcement

If the bending moment is sagging at the column points reinforcement is to be provided at bottom and if the bending moment is hogging at the span

reinforcements are to be provided at the top. But in both cases minimum reinforcement is to be provided at the opposite faces as the thickness of the raft is very high.

With the same procedure required reinforcement can be calculated for all other strips parallel to x-direction. To calculate the reinforcement for the strips parallel to y-direction all vertical column loads and the concentrated column moments about x-direction are to be considered. Other things for calculation are same as given in the steps stated above.

In case of steel super-structure during design of raft the column loads are increased by ten percent to take care of the weight of the pedestal. Other design procedure is same as stated above.

Check Against Shear

For each column which is supported by the raft two way shear, i.e., punching shear test is must. Critical section for punching shear is at a distance of $d_3/2$ from the face of the column. Here $d_3$ is the effective depth of the raft. Shear stress can be calculated by

$$\tau_{cal} = \frac{P_i}{l_{pe}\, d_3}$$

where,

$P_i$ = vertical load on $i_{th}$ column

$l_{pe}$ = periphery of the area at a distance of $(d_3/2)$ from the column face.

For interior columns

$$l_{pe} = 2\,(w_{cx} + w_{cy} + 2\,d_3) \tag{2.16}$$

For side columns, if $d_3/2$ is greater than the edge distance from the face of the column then

$$l_{pe} = 2(w_{cx} + d_3) + (w_{cy} + d_3/2)$$

or

$$l_{pe} = 2(w_{cy} + d_3) + (w_{cx} + d_3/2)$$

Otherwise, Eq. 2.16 is to be used. For corner columns, if $d_3/2$ is greater than the edge distance from the two perpendicular faces of a corner of a column then

$$l_{pe} = (w_{cx} + w_{cy} + d_3)$$

.

Otherwise, Eq. 2.16 is to be used. This $\tau_{cal}$ must not exceed $k_s\ \tau_c$ [IS:456, Clause 30.6.3.1]. where,

$$k_s = (0.5 + \beta_c)$$

$$k_s \not> 1.0$$

$$\beta_c = \frac{short\ side\ of\ the\ column}{long\ side\ of\ the\ column}$$

$\tau_c = 0.25\sqrt{f_{ck}}$ for Limit State method of design. All columns have to satisfy this condition.

## 2.5 FORMULATION OF MATHEMATICAL PROGRAMMING

Determination of minimum value of objective function subject to constraints as stated earlier can be stated as a generalized optimization problem as follows :

Find $\mathbf{D}_m$ such that

$F(\mathbf{D}_m)$ is minimum

subject to $g_j(\mathbf{D}_m) \leq 0.0$

where, $\mathbf{D}_m$ is a function of design variables

$g_j(\mathbf{D}_m)$ are constraints.

The problem is converted into an unconstrained optimization problem with the help of Extended Penalty Function Method (Kavlie and Moe, 1971). The Sequential Unconstrained Minimization of the developed composite function is then carried out in conjunction with Powell's Conjugate Direction Method for multidimensional search with Quadratic Interpolation Technique for linear minimization. The composite function $\phi(\mathrm{D}, r_k)$ developed by blending the objective function and constraints is as follows :

$$\phi(\mathrm{D}, r_k) = F(\mathrm{D}) + r_k \sum_{j=1}^{M} G[g_j(\mathrm{D})]$$

The function $G[g_j(\mathrm{D})]$ is chosen as

$$G[g_j(\mathrm{D})] = \begin{cases} 1/g_j(\mathrm{D}) & \text{when } g_j(\mathrm{D}) \leq \epsilon \\ \frac{2\,\epsilon - g_j(\mathrm{D})}{\epsilon^2} & \text{when } g_j(\mathrm{D}) > \epsilon \end{cases}$$

where, $\epsilon = -r_k/\delta$

$r_k$ = penalty parameter and $\delta$ = transition parameter.

Advantage of this method is that even if the initial design vector is in the infeasible region the algorithm is capable of finding the feasible optimum solution. For various engineering problems most of the times initial feasible design vector is difficult to guess. In such cases, the developed technique is expected to be helpful in finding a feasible solution.

### 2.5.1 Convergence Criteria

If the relative change of objective function and design variables between two consecutive cycles satisfy the following conditions then the convergence is assumed to be achieved.

$$\left|\frac{F_k - F_{k-1}}{F_k}\right| \leq \epsilon_1 \tag{2.17}$$

$k$ = number of iterations

$F$ = objective function.

$\epsilon_t$ = very small numerical quantity.

# Chapter 3

# RESULTS AND DISCUSSIONS

A program has been developed in FORTRAN language to obtain the optimal solution following the procedure outlined in the previous chapter. All computations have been carried out using Pentium processor. Effectiveness of the suggested method has been demonstrated through parametric study using different soil parameters (like $c$, $\phi$, $E_s$, $\mu$, $C_c/(1+e_o)$ ).

To estimate the total cost of the raft the rates used for concrete, steel and other construction items are according to the Central Public Works Department's schedule rate for the year 1997. The rates are presented in the Table 3.1. The characteristic strength of concrete and steel ($f_{ck}$ and $f_y$) are taken as $20N/mm^2$ and $415N/mm^2$ respectively.

Using the developed program it is possible to design a raft on any type of soil. The soil may be purely cohesive, purely frictional or a general cohesive-frictional soil. It can also be used when the soil data is available in terms of CPT value with depth for the considered site. The permissibe value of settlement has been taken to be $50mm$ if it is not specified otherwise.

For a given set of loading shown in Figure 3.1 parametric study has been carried out and the obtained results are as follows.

The developed computer program has been varified by computing the

Table 3.1: Rate of the items in Rs/$m^3$

| Particulars | rates |
|---|---|
| cost of excavation | 15 |
| cost of concrete | 1230 |
| cost of steel | 1670 |
| cost of filling | 8 |

obtained solution for different strips with manually computed values. The problem and the results obtained by computer program and by the hand calculation are shown in the Table 3.2. The results show good agreement with each other.

The influence of the initial design vector on the optimal solution has been studied. The results are presented in Table 3.3. Soil condition is same as given in Table 3.2. It can be seen from the table that starting from widely differing initial design vectors the optimal vectors is practically identical and the costs differ only marginally (0.2 %). So, the optimal solution can be considered to be the global solution. Convergence of the numerical scheme with decreasing sequence of the penalty parameter $r_k$ is shown in Fig. 3.2. It can be seen that as $r_k$ decreases the composite function and the objective function tend to converge for $r_k$ less or equal to 20 showing that minimization of the composite function led to the minimization of the objective function. From Fig. 3.3 it is seen that as the number of function evaluations exceeds about 350 both the composite function and objective function converge. It is observed from Fig. 3.4 that when the number of iterative cycles exceeds 2 the convergence is gradual and ex-

Table 3.2: Comparison between hand-calculation and program-obtained value

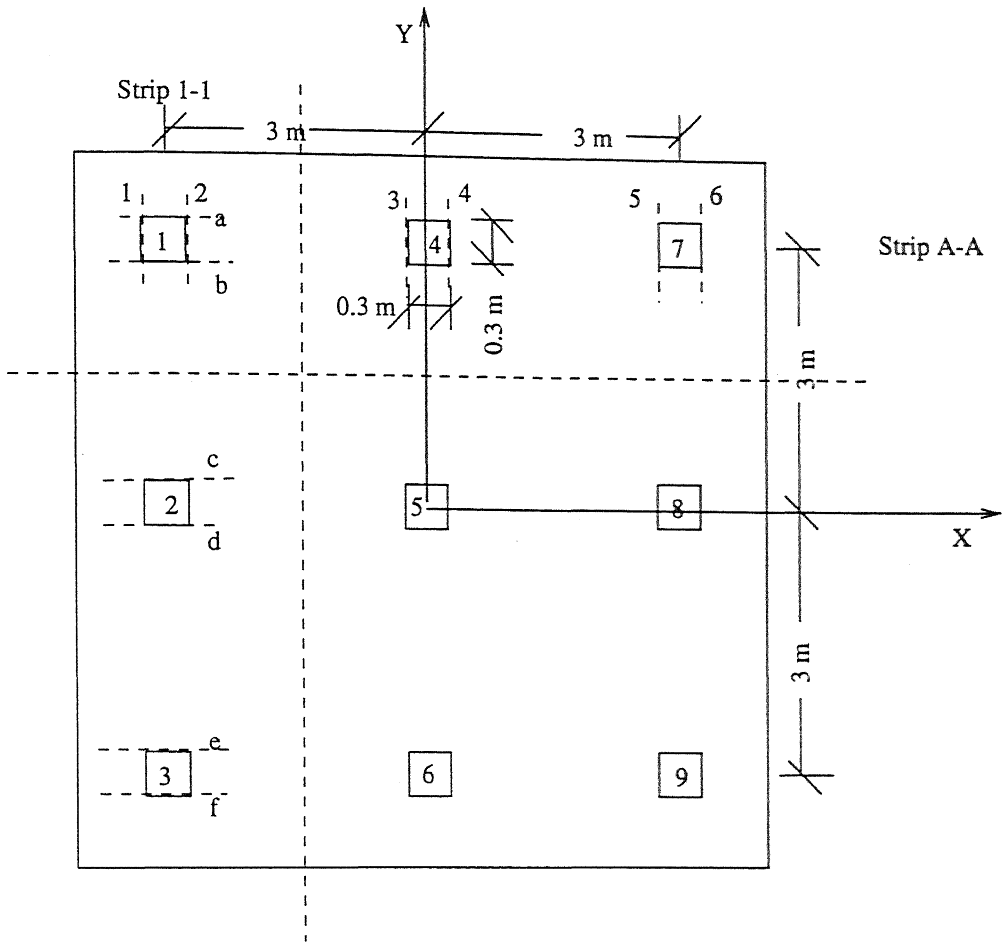
$\phi = 0$
Es = 40 MPa
$\mu = 0.45$
$Cc/(1+e_0) = 0.05$
GWT at 0.0 m depth
$\gamma = 20$ kN/m$^3$

| strips | points | B.M. from program | B.M. from hand calculation |
|---|---|---|---|
| A-A | 1 | 26.72 | 26.718 |
| | 2 | 52.29 | 53.24 |
| | 3 | -47.68 | -47.566 |
| | 4 | -24.21 | -24.358 |
| | 5 | 2.89 | 3.099 |
| | 6 | 41.59 | 41.092 |
| 1-1 | a | 36.82 | 36.829 |
| | b | -3.940 | -3.686 |
| | c | -37.164 | −37.312 |
| | d | -72.876 | -73.385 |
| | e | 13.284 | 12.338 |
| | f | -0.849 | -1.069 |

Table 3.3: Effect of initial variables on the result

$\phi = 0$
Es = 40 MPa
$\mu = 0.45$
$Cc/(1+e_0) = 0.05$
GWT at 0.0 m depth
$\gamma = 20$ kN/m$^3$

| Initial variables (m) | | | | Final variables (m) | | | | Cost (Rs) |
|---|---|---|---|---|---|---|---|---|
| $d_1$ | $d_2$ | $d_3$ | $d_4$ | $d_1$ | $d_2$ | $d_3$ | $d_4$ | |
| 1 | 1 | 1 | 0.02 | 0.15 | 0.15 | 0.75 | 0.012 | 48577 |
| 4 | 4 | 2 | 0.6 | 0.154 | 0.150 | 0.75 | 0.0132 | 48680 |




| Column no. | P | Mx | My |
|---|---|---|---|
| 1 | 250 | -40 | 40 |
| 2 | 250 | -20 | 30 |
| 3 | 300 | -40 | 30 |
| 4 | 200 | -50 | 30 |
| 5 | 200 | -20 | 40 |
| 6 | 250 | -40 | 30 |
| 7 | 200 | -30 | 40 |
| 8 | 250 | -40 | 20 |
| 9 | 250 | -20 | 40 |

Figure 3.1: Sample loading from the super-structure

cellent. These figures show that the adopted method is quite effective in isolating the optimal solution.

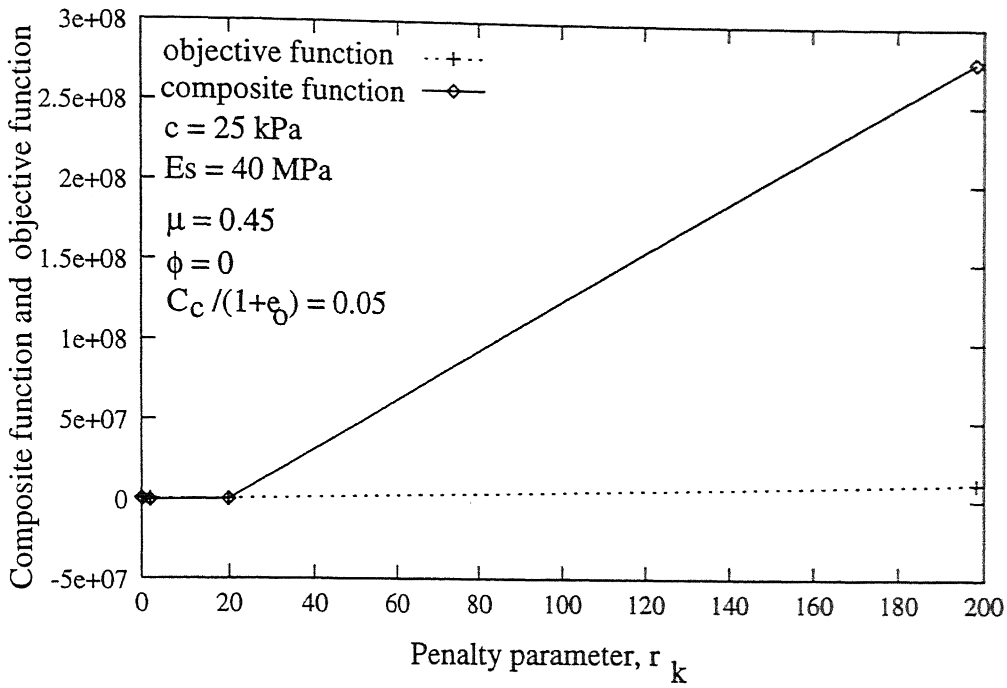



Figure 3.2: Convergence of objective function and composite function with respect to Penalty parameter

The stress and settlement calculations have been carried out by discretizing the plan area of the raft into several small elements and taking the effect of each element at each corner of the raft. Equations 2.4 and 2.5 have been used to calculate stress and settlement. As the application of Boussinesq's equation cannot take into account the effect of the embedment of the foundation in the soil medium, the results have been checked by using Mindlin's solution. The comparative results are shown in Table 3.4. It can be seen from the table that the results do not vary much (0.05 % only). For all the cases value of $E_s$ is $50000 kN/m^2$. This shows that for the given problem the embedment of the foundation does not play any significant role on the overall cost and so also the other parameters like $\mu$,

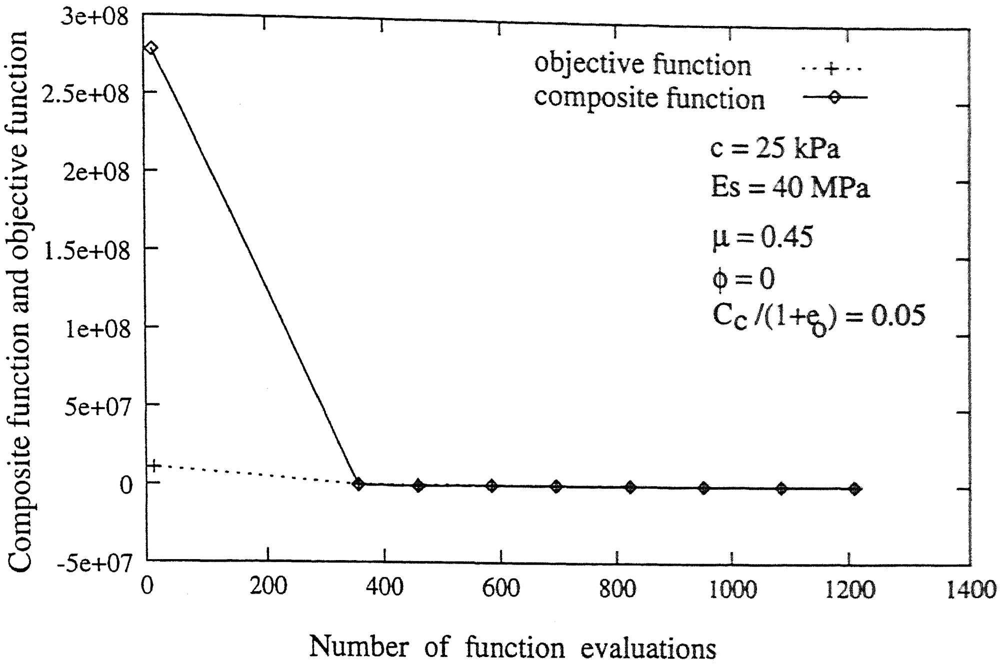



Figure 3.3: Convergence of objective function and composite function with respect to Number of function evaluations

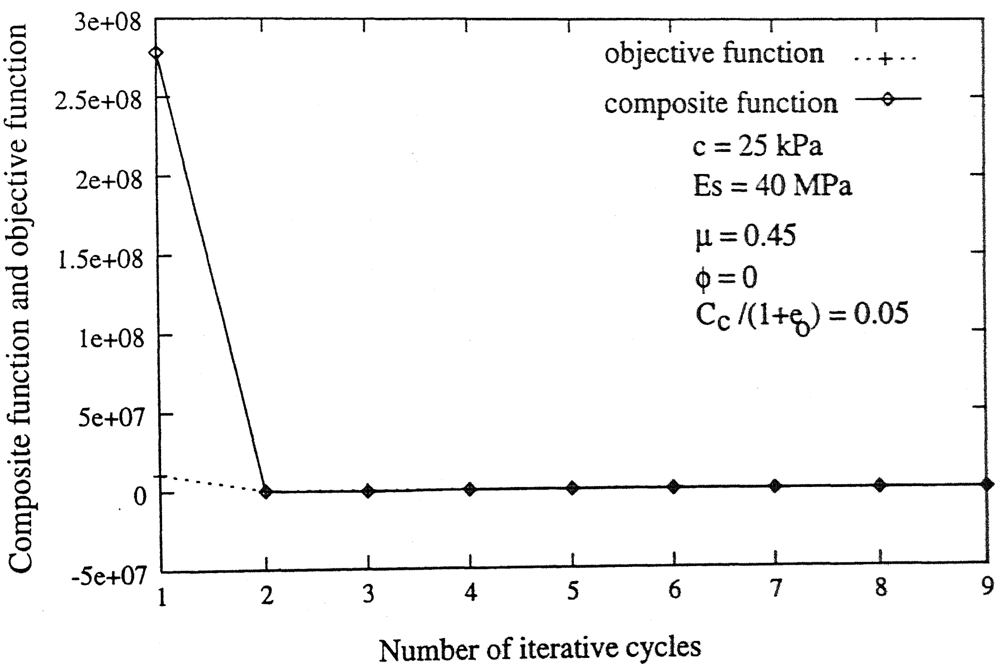



Figure 3.4: Convergence of objective function and composite function with respect to number iterative cycles

Table 3.4: Effect of Boussinesq's and Mindlin's solution on the results

| $S_{max}$ in mm | $c(kN/m^2)$ | $\phi$ (in degree) | $\mu$ | $C_c/(1+e_o)$ | cost in Rs by Boussinesq | cost in Rs by Mindlin |
|---|---|---|---|---|---|---|
| 50 | 20 | 10 | 0.45 | 0.06 | 48557 | 48556 |
| 50 | 20 | 15 | 0.45 | 0.06 | 48545 | 48586 |
| 50 | 20 | 20 | 0.45 | 0.06 | 48524 | 48530 |
| 25 | 10 | 30 | 0.3 | 0.05 | 48557 | 48505 |
| 50 | 10 | 30 | 0.3 | 0.05 | 48521 | 48522 |
| 75 | 10 | 30 | 0.3 | 0.05 | 48542 | 48565 |
| 100 | 10 | 30 | 0.3 | 0.05 | 48543 | 48565 |

$C_c/(1+e_o)$, $\phi$, $c$ and settlement limit. This is due to the fact that for these paramaters settlement constraints are inactive.

A study is made to estimate the ammount of saving that can be achieved over a manual design. For this a few trials are made and the best out of these is fed to the computer as the initial design vector. At the first iteration it gives similar results to the values obtained by hand calculation. After 9 iterations final optimal results are obtained. The results are shown in Table 3.5. It can be seen that at the optimal point the settlement constraints are all active and so also some geometric and bearing capacity constraints. A net saving of $Rs11,892$/- can be obtained which is about 19.65 % of the best possible cost that has been acieved manually. This demonstrates the necessity of adopting optimization procedure to get the mimimum cost design.

**Parametric Study**

For saturated fine grained soils under undrained condition ($\phi$=0), the effect of the undrained shear strength ($S_u$) on the overall cost of the raft has been studied by varying the $S_u$ value ranging from 20 $kPa$ to 40 $kPa$. The results are presented in Fig. 3.5. It is observed from the figure that when

Table 3.5: Comparison between best hand-calculated result and optimum result for same soil and loading condition

| |
|---|
| Soil condition :<br>$c$ = 10 kPa<br>$\phi = 15^{0}$<br>Es = 25 MPa<br>$\mu$ = 0.499<br>GWT at 0.0 m depth<br>$\gamma$ = 20 kN/m$^3$<br>$C_c/(1+e_o)$ = 0.07 |
| Initial variables ( $d_1, d_2, d_3, d_4$ )<br>0.5, 0.5, 0.75, 0.016<br>Initial area ratio (Ar)<br>25.724<br>Initial constraints ( first 8 constraints are geometric constraints, next 4 are differential settlement constraints, next 8 are bearing capacity constraints and last 4 are total settlement constraints)<br>-0.5, -0.5, 0.0, -1.25, -0.35, -0.35, -0.0040, -0.0777<br>0.0034.38, -0.0030, -0.00331, -0.0033, -659..933, 699.816,<br>-645.239, -645.239, -134.26, -94.3768, -188.837, -148.954, -0.014,<br>-0.0155, -0.0125, -0.0109<br>Initial cost<br>Rs 60518 |
| Final variables ( $d_1, d_2, d_3, d_4$)<br>0.151, 0.155, 0.75, 0.012<br>Final area ratio(Ar)<br>1.806<br>Final constraints ( first 8 constraints are geometric constraints, next 4 are differential settlement constraints, next 8 are bearing capacity constraints and last 4 are total settlement constraints)<br>-0.8482, -0.8447, -0.000054, -1.2499, -0.00174, -0.005277,<br>-0.000411, -0.0813, -0.00354, -0.00294, -0.00334, -0.003421,<br>-722.765, -777.384, -648.274, -702.893, -160.154, -105.535,<br>-234.645, -180.025, -0.00965, -0.00115, -0.00774, -0.00655<br>Final cost<br>Rs 48626 |

$S_u \geq 30\ kPa$ the undrained shear strength value does not have any influence on the cost of the raft. The influence of the undrained shear strength value on the cost is appreciable for strength value less than $30\ kPa$. As the the strength value is decreased from $30\ kPa$ to $20\ kPa$, the rate of increase in the cost increases at an increasing rate. Rate of increase in cost is 9 % when $S_u$ is decreased from $30\ kPa$ to $25\ kPa$ (i.e., 16.7 % decrease in strength ) and 20 % when it is further decreased to $20\ kPa$ from $25\ kPa$ (20 % decrease in strength). This increase in the cost when the strength value decreases can be justified from the study presented in the Fig. 3.6 where the influence of the varying strength value on the area of the raft has been shown. To do this, a parameter 'area ratio' ($A_r$) has been introduced

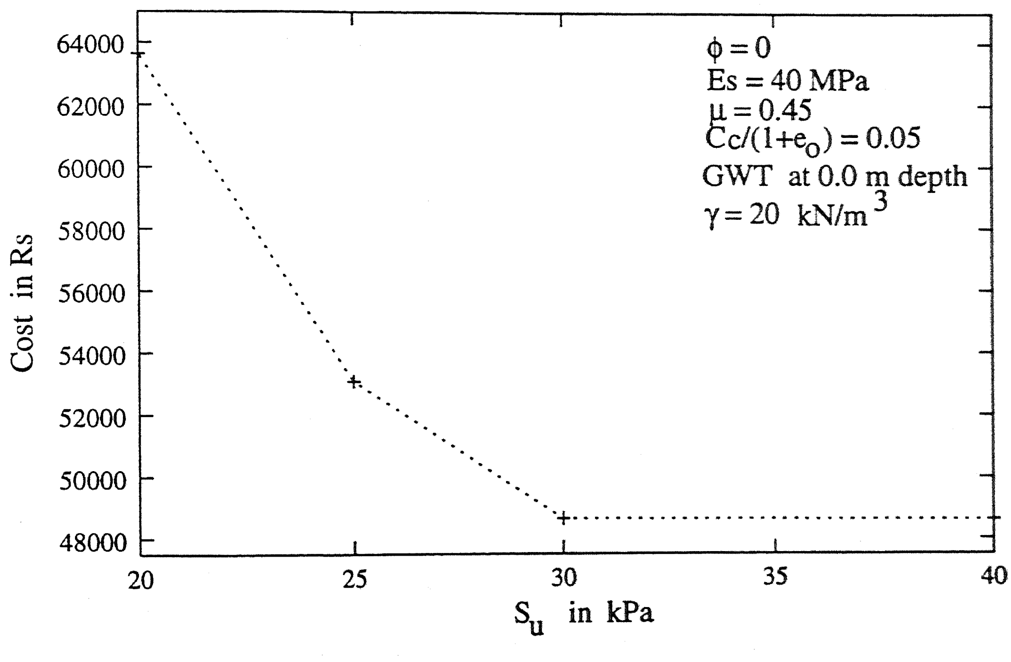



Figure 3.5: Variation of cost with $S_u$

which is given by the following formula:

$$A_r = \frac{A_1 - A_o}{A_o} \times 100$$

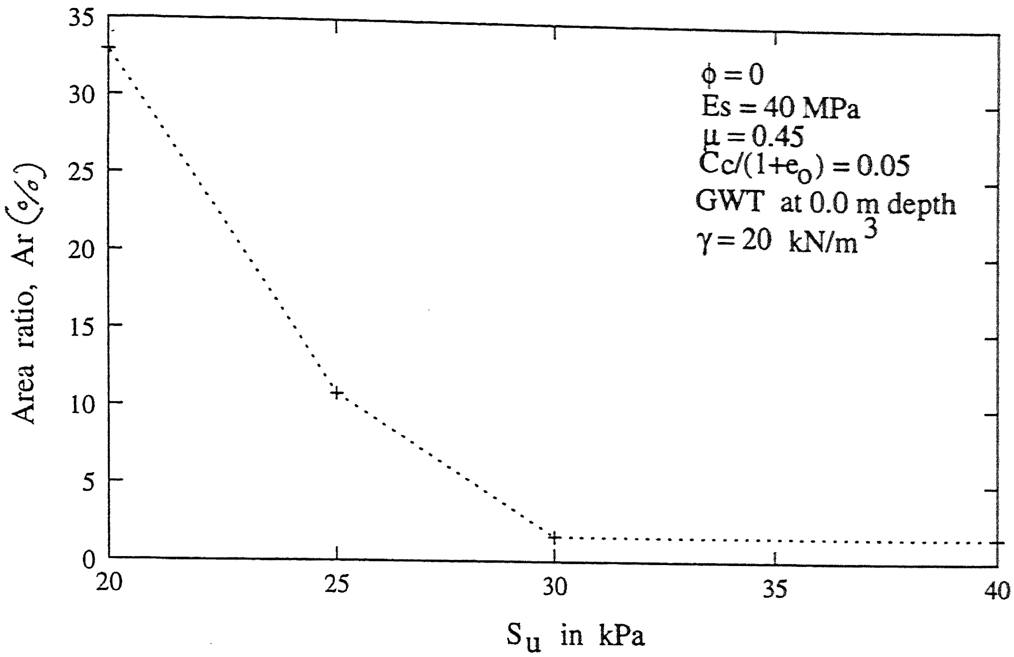



Figure 3.6: Variation of 'area ratio' $A_r$ with $S_u$

where,

$A_o$ = base area of the super structure

$A_1$ = area of the raft

It can be seen from the figure that as strength value decreases the change in $A_r$ is not significant till the strength value does not fall below 30 $kPa$; below this value the increase in the 'area ratio' is very sharp for decreasing values of $S_u$. It can be seen that the increase in 'area ratio' is 9 % when strength value decreases from 30 $kPa$ to 25 $kPa$ while the same is 22 % when the strength value is decreased to 20 $kPa$ from 25 $kPa$. As $S_u$ is decreased below 30 $kPa$ the bearing capacity value decreases resulting in the increase area of the raft and consequent increase in the cost. When $S_u$ is greater than 30 $kPa$, the bearing capacity does not control the design. It is interesting to note that over the whole range of $S_u$ values varying from 20 $kPa$ to 40 $kPa$ the depth of the raft remains almost constant throughout

(Fig. 3.7). This signifies that the increase in cost is only due to the increase in plan area of the raft. From Fig. 3.8 one can observe that the cost of concrete predominantly influence the cost of the raft. Over a wide range of undrained shear strength the ratio of the cost of concrete and cost of steel almost remain same and the ratio is about 5.2 to 5.3. So to save the cost one can design the raft as a flexible one which will give a low value thickness resulting in ultimate savings in the cost of concrete. However, in such a case the effect of differential settlement at the different column points have to be taken into account on the overall behaviour of the structure. Therefore, one must adopt integrated design procedure considering the super-structure and foundation as a single unit.

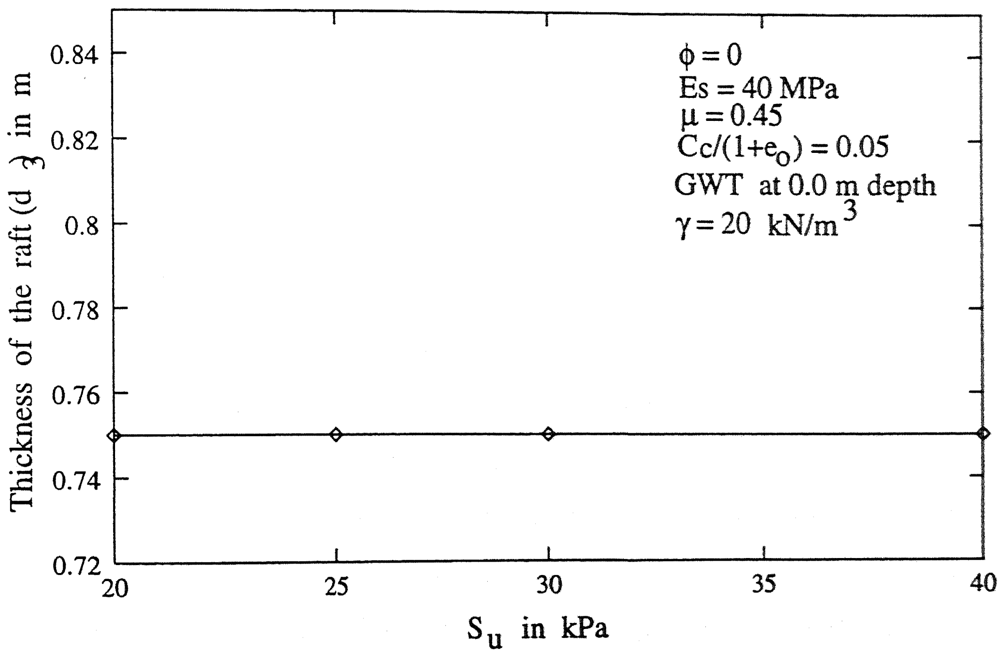



Figure 3.7: Variation of thickness of the raft ($d_3$) with respect to $S_u$

From Fig. 3.9 it can be seen that for purely cohesionless soils for given values of $\phi$ and other parameters, the variation of $\mu$ over a range of 0.35 to 0.499 does not play any significant role on the cost as well as on the optimal

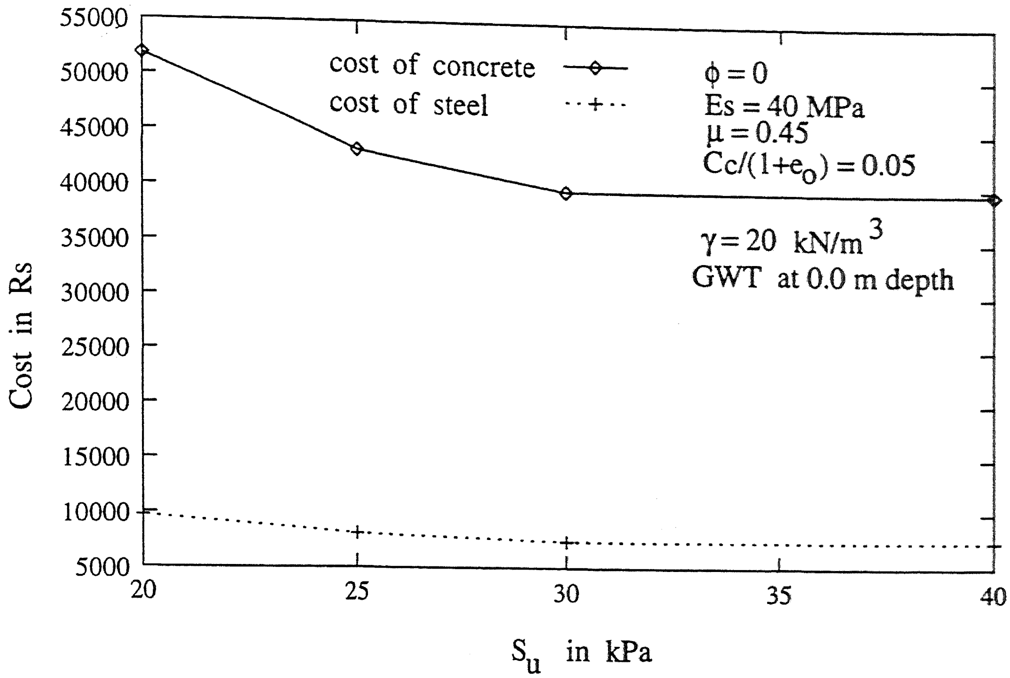



Figure 3.8: Variation of cost of concrete and steel with respect to $S_u$

dimensions as the design is not controlled by settlement. Settlement is always less than the permissible value. But, from Fig. 3.10 it can be noticed that for $E_s$ below 25 $MPa$ effect of $E_S$ on the raft is greater. At this stage design is strictly controlled by settlement and plan area of the raft is highly increased resulting a high 'area ratio' ($A_r$). But, the depth of the raft did not change in any case. This is due to the fact that thickness required from shear and bending moment criteria is much less than the geometric constraint ($d_3 \geq 0.75m$) and, as such, this constraint decomes active during optimization. For the sake of brevity all the results are not presented here.

From Fig. 3.11 it can be seen that for the given set of soil parameters, $C_c/(1+e_o)$ does not have any significant effect on the cost, till it reaches the limiting value of settlement up to which the design is not governed by the settlement criteria. But, as the value of $C_c/(1+e_o)$ increases the rate

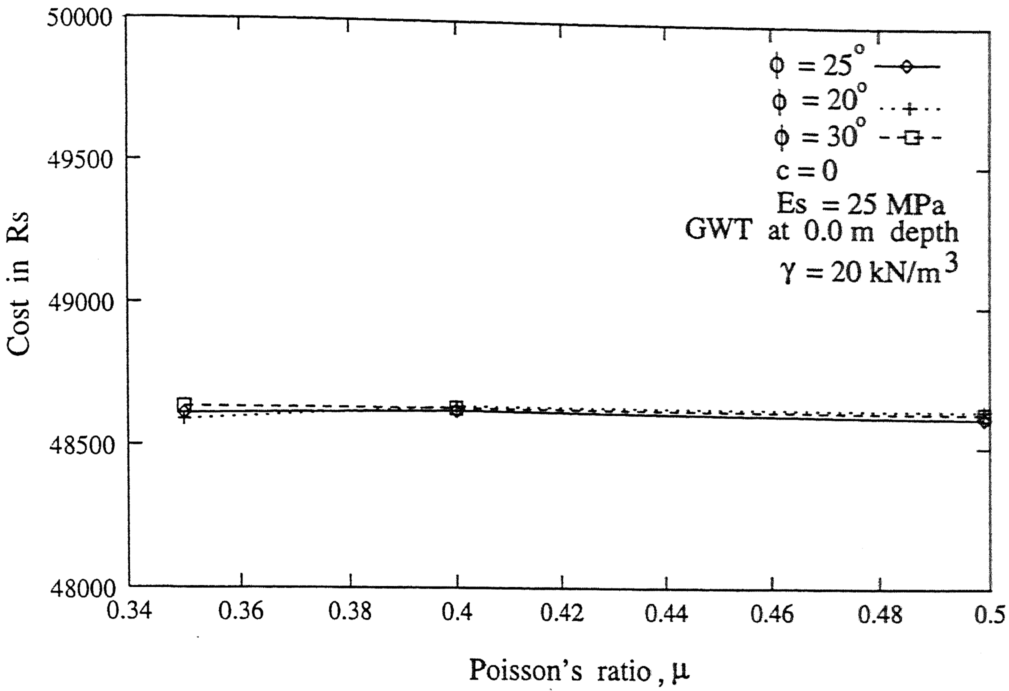



Figure 3.9: Variation of cost with respect to $\mu$ for different values of $E_s$

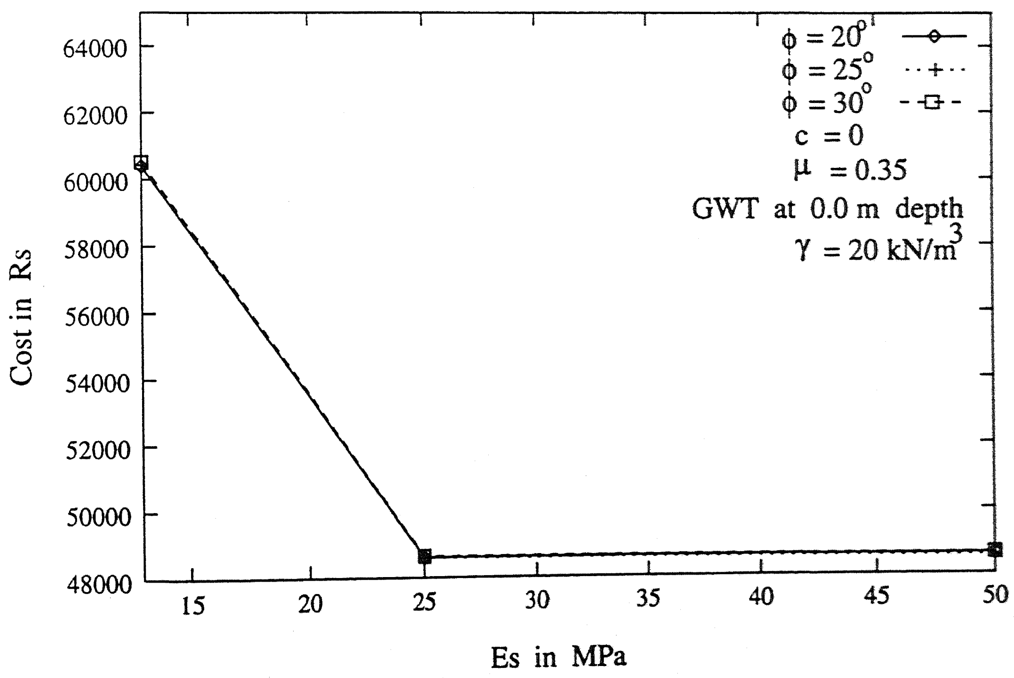



Figure 3.10: Variation of cost with respect to $E_s$ for different values of $\phi$

of settlement also increases and when it reaches the limiting value ($50mm$) the design is fully controlled by settlement. The variation of obtained 'area ratio' due to the change in $C_c/(1+e_o)$ are shown in Fig. 3.12. It is seen from this figure that for $E_s$ equal to 25 $MPa$ the result is not affected much and the settlement reaches nearer to the limiting value ($50mm$) as the design is then fully controlled by settlement. In Fig. 3.12, for $E_S$ equal to 25 $MPa$ the result is not affected much till the value $Cc/(1+e_o)$ reaches 0.105 but when it is 0.114 the cost of the raft is very high. Similarly, for $E_s$ equal to 50 $MPa$, up to the value of $C_c/(1+e_o)$ equal to 0.15 the cost of the raft does not change. When it is increased to 0.154 cost is increased by about three thousand rupees. But, the cost is maximum when it reaches 0.164 beyond which no solution could be formed. But, there is no change in the obtained thickness of the raft, which is only $0.75m$.

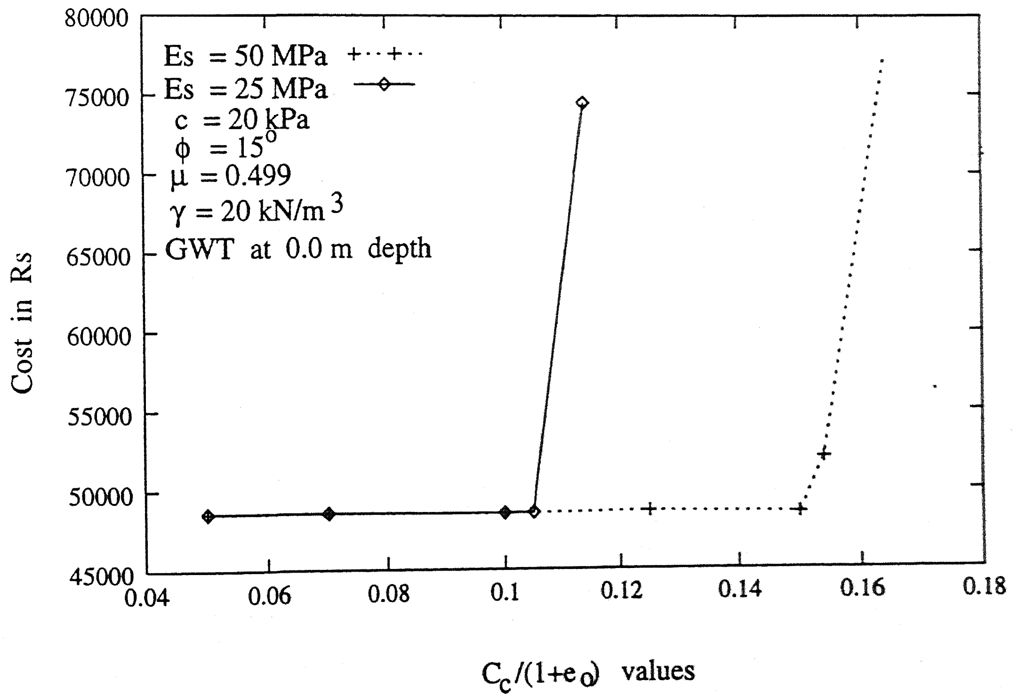



Figure 3.11: Variation of cost with respect to $C_c/(1+e_o)$ for different values of $E_s$

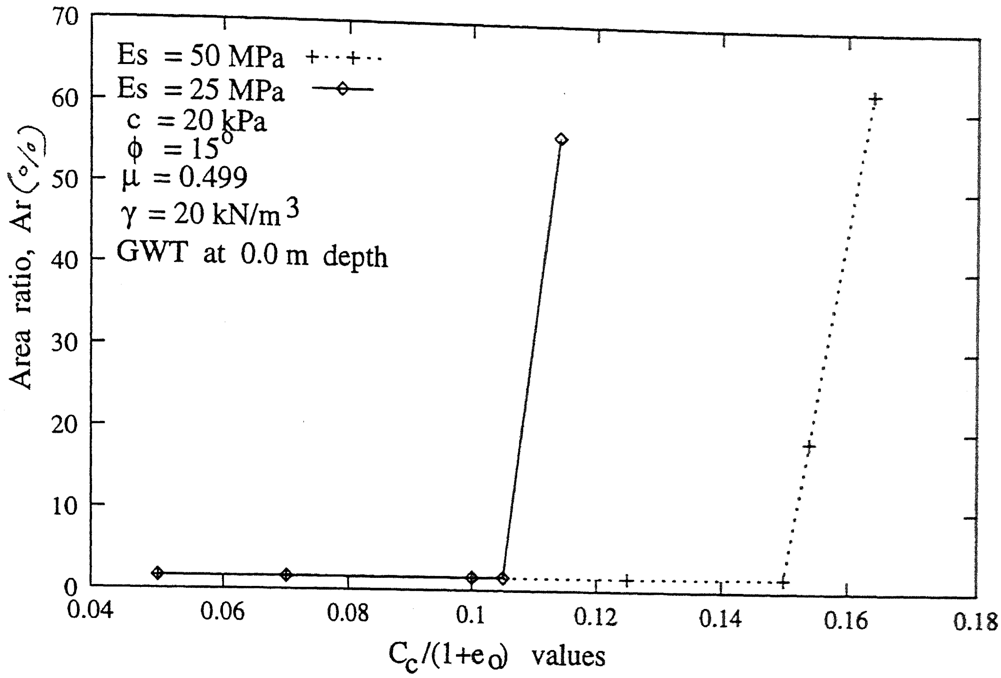



Figure 3.12: Variation of 'area ratio' ($A_r$) with respect to $C_c/(1+e_o)$ for different values of $E_s$

To see the effect of permissible-limit of settlement on the optimum cost, four limiting values of settlement i.e., 25$mm$, 50$mm$, 75$mm$ and 100$mm$ have been chosen. For achieving enhaunced settlement the column loads are increased sequentially by some percentage. The results are presented in Table 3.6. It is seen that only for 5 % increase in the initial loading as given in Fig. 3.1, there is a great increase in the cost when maximum settlement is limited to 25$mm$. At the same loading condition if the settlement limit is increased to 50$mm$ and 75$mm$ the optimal cost is reduced to a minimum level. No further change in permissible limit of settlement causes any change in the cost. Similar behaviour is observed when the permissible settlement limit is increased to 75$mm$ from 50$mm$ and the raft can take 280 % of the initial loading. When the settlement limit is increased to 100$mm$ from 75$mm$, the raft can carry 450 % of the initial loading. Table 3.6 shows

the effect of maximum settlement limit on the optimum cost as well as on

Table 3.6: Results for different maximum limiting settlements

| Soil Condition : $c$ = 10 kPa, $\phi$ = 15°, $\mu$ = 0.499 | Es = 50 MPa, $C_c/(1+e_o)$ = 0.07 | $\gamma$ = 20 kN/m$^3$, GWt at 0.0 m depth | | |
|---|---|---|---|---|
| | Optimum Results for Maximum Settlement Limits | | | |
| Loading (multi--ple of loading given in Fig. 3.1) | Smax = 25 mm | Smax = 50 mm | Smax = 75 mm | Smax = 100 mm |
| 1.05 | cost Rs 50937<br>Ar = 6.53<br>$d_3$ = 0.75 m<br>$d_4$ = 12 mm | cost Rs 48591<br>Ar = 1.576<br>$d_3$= 0.75 m<br>$d_4$= 12 mm | cost Rs 48555 | Cost Rs 48555 |
| 2.8 | no solution | cost Rs 71027<br>Ar = 46.903<br>$d_3$ = 0.75 m<br>$d_4$ = 12 mm | cost Rs 48627<br>Ar = 1.592<br>$d_3$= 0.75 m<br>$d_4$= 12 mm | cost Rs 48661 |
| 4.5 | no solution | no solution | cost Rs 73210<br>Ar = 53.50<br>$d_3$ = 0.75 m<br>$d_4$ = 12 mm | cost Rs 48815<br>Ar = 1 542<br>$d_3$= 0.75 m<br>$d_4$= 12 mm |
| 6.5 | no solution | no solution | no solution | cost Rs 74130<br>Ar = 55.49<br>$d_3$ = 0.75 m<br>$d_4$ = 14 mm |

the 'area ratio' ($A_r$). Here also, the thickness of the foundation does not affect the cost and the value always being at its lower limit, i.e., $0.75m$. In all the cases, the high cost, can be attributed to the setlement limit and corresponding increased 'area ratio' ($A_r$).

The program can also be used if the raft is placed soil strata for which

CPT data is available. To demonstrate its use, the test data reported by Singh (1982) for IIT Kanpur Campus has been used (Fig. 3.13). The value

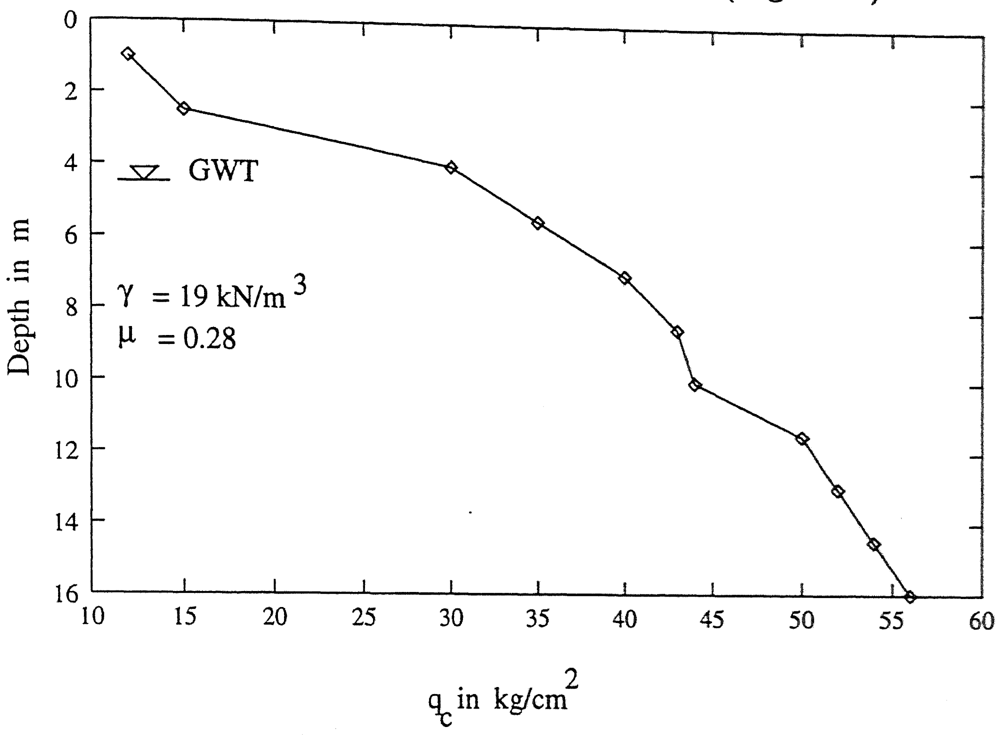



Figure 3.13: Variation of cone resistance with depth

of $\alpha$ for Campus soil is 9 (Prakash Rao, 1985). Using Equation 2.8, $E_s$ is calculated for different $q_c$ and using Schmertmann Method total settlement is calculated. Obtained optimal solution is given in Table 3.7.

Table 3.7: Results for layered cohesionless soil

| optimum dimensions in m | optimum cost in Rupees | area ratio |
|---|---|---|
| 6.3 X 6.3 X 0.75<br>Final vector<br>0.15, 0.15, 0.75, 0.012 | 48526 | 1.581 |

In Fig. 3.14 reinforcement detailing for the above case is shown.

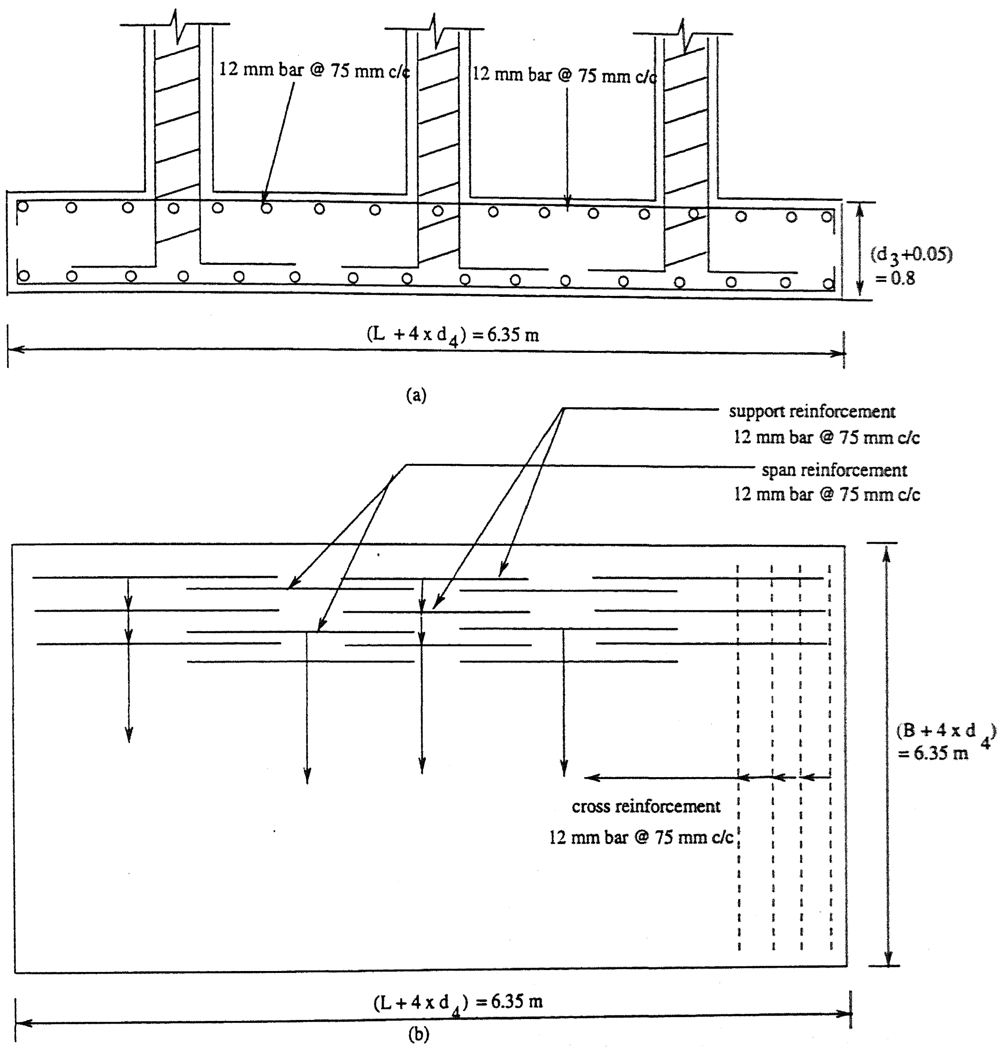



Figure 3.14: Reinforcement detailing for a raft, (a) sectional view, (b) plan

To see the effect of loading on the dimension of the raft, the load on the central column is increased and the obtained value of 'area ratio' $A_r$ is plotted against loading. In Fig. 3.15 and 3.16 these effects are shown. It

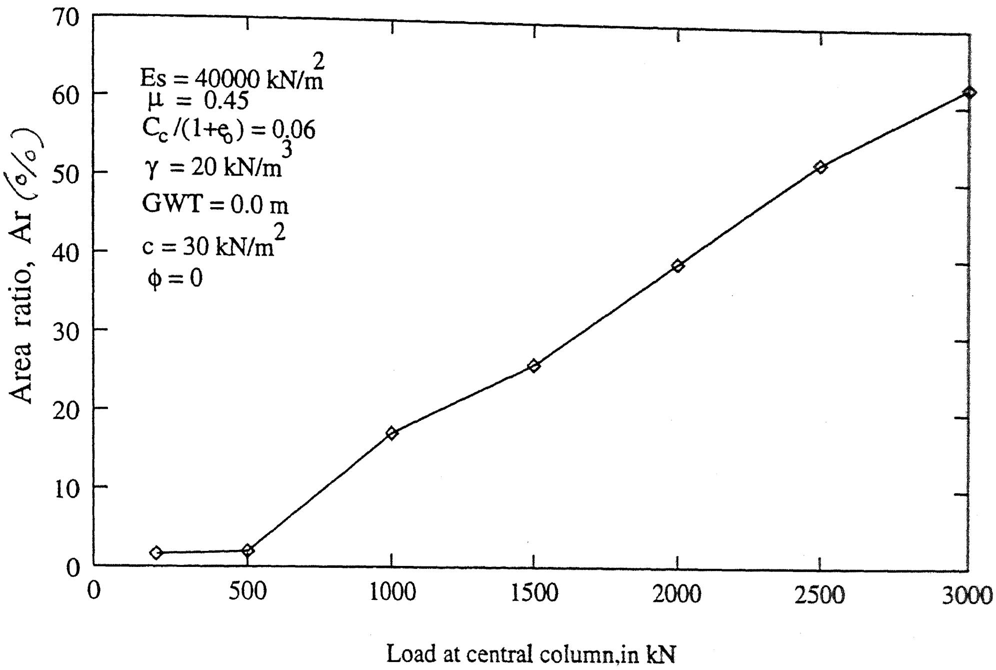



Figure 3.15: Effect of increased loading on 'area ratio' $A_r$, for $c$-soil

can be observed from the figures that for $\phi = 0$ case the effect of increase in load on the central column is more pronounced than that in $c-\phi$ -soils. The thickness of the raft is again the same ( $0.75m$ ) for all the cases and equal to the lower limit imposed on the depth of the raft.

For the same set of loading the effect of spacing of the columns on the area of the raft has also been studied. It can be seen from Fig. 3.17 that for $\phi = 0$ case when the spacing is less than $3m$ the effect is highly pronounced and the 'area ratio' $(A_r)$ suddenly increases. This is due to the bearing capacity requirement. If the spacing is less then total base area of the super-structure decreases, and to get required bearing capacity area of the raft increases. But, for general $c-\phi$ -soil $A_r$ does not vary much with the

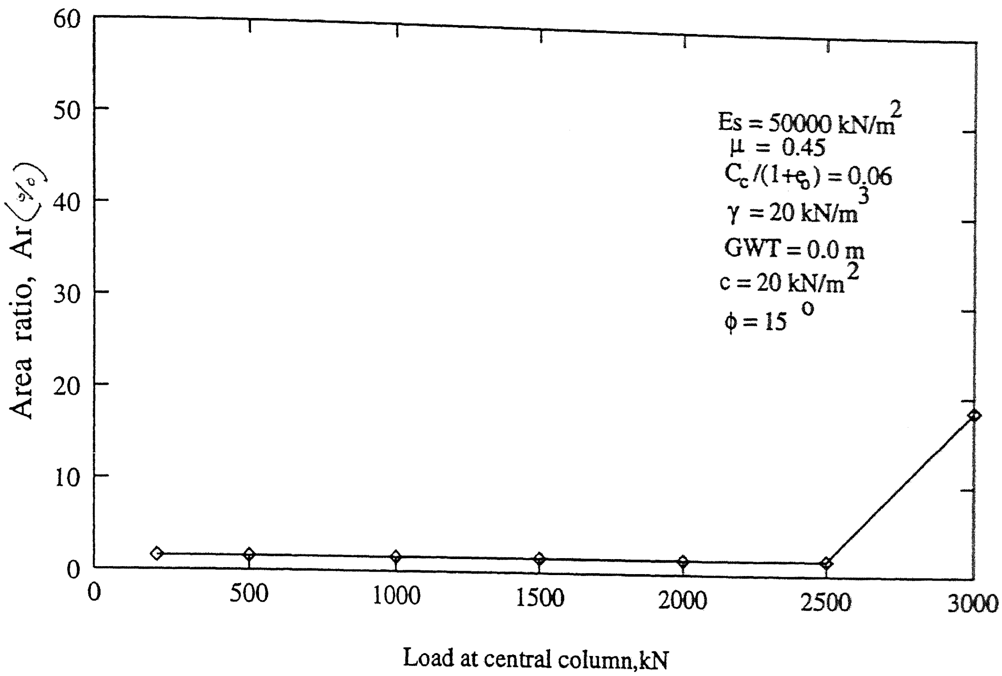



Figure 3.16: Effect of increased loading on 'area ratio' $A_r$, for $c - \phi$ -soil

increase of the spacing. It is shown in Fig. 3.18. However this observation may be problem specific.

The results discussed till now are for designing the raft for concrete super-structure. In the program there is a provision to design a raft for the steel super-structure. For the same loading and for only cohesive soil a sample problem has been solved. In Table 3.7 comparison between the results obtained for steel super-structure and concrete super-structure has been produced for same soil condition as shown. It can be observed from the results that cost for steel structure is more and the additional cost can be attributed to the cost of the pedestals.

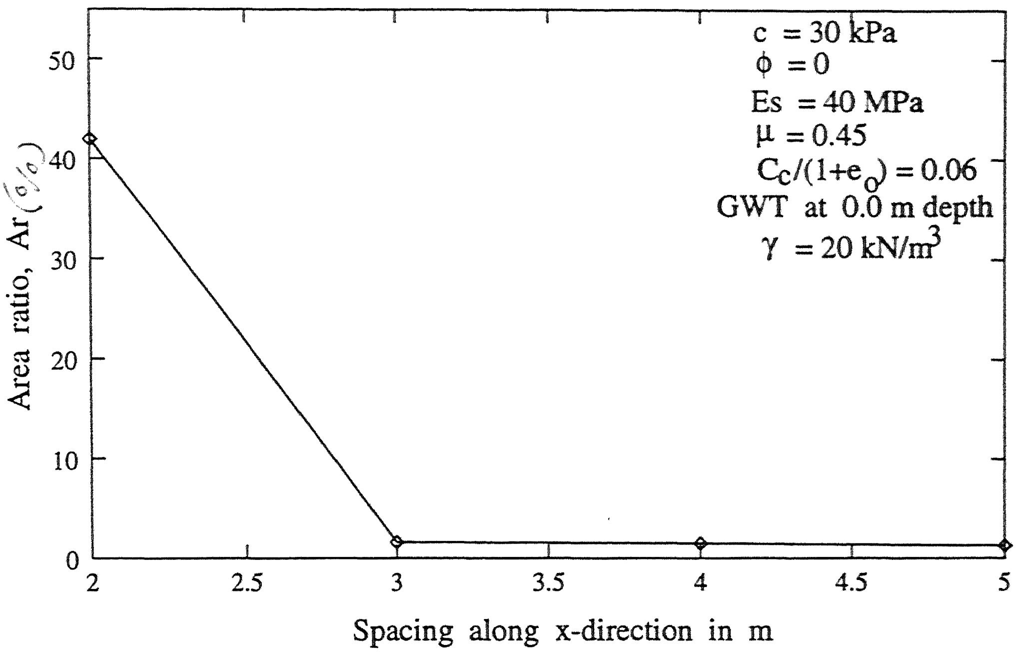



Figure 3.17: Effect of spacing of the columns on 'area ratio' $A_r$, for $c$-soil

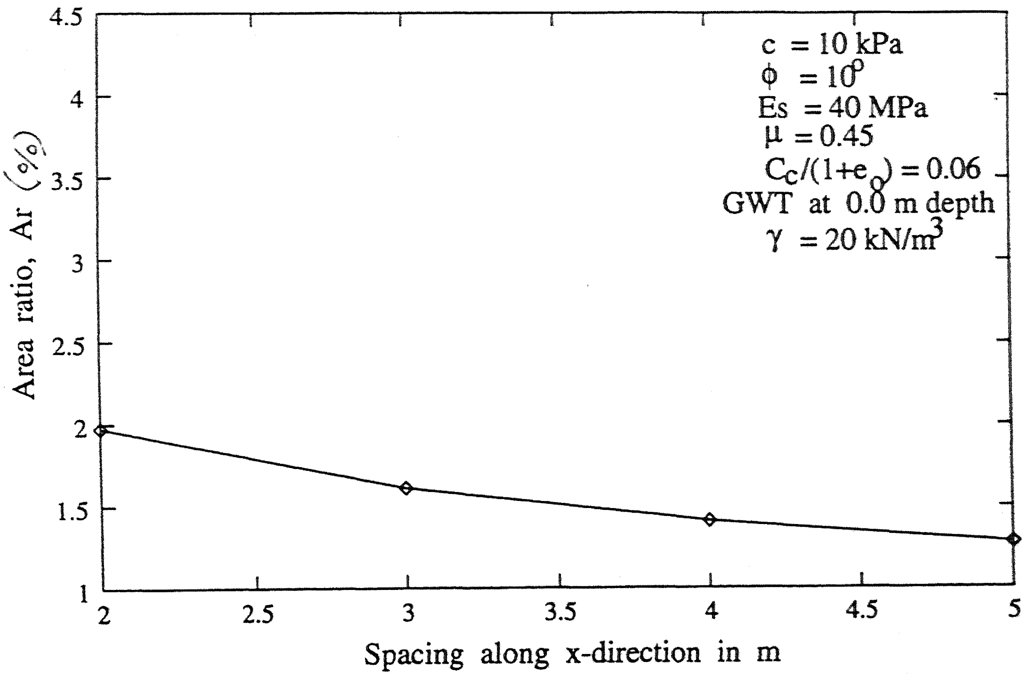



Figure 3.18: Effect of spacing of the columns on 'area ratio' $A_r$, for $c - \phi$ -soil

Table 3.8: Comparative results for steel and concrete super-structure

Soil conditions:
$c = 20$ kPa, $\phi = 0$, GWT at 0.0 m depth
$E_s = 40$ MPa, $\mu = 0.45$
$C_c/(1+e) = 0.05$, $\gamma = 20$ kN/m$^3$

| Particulars | Steel super-structure | | Concrete super-structure | |
|---|---|---|---|---|
| Design vectors | Initial | Final | Initial | Final |
| d1 (m) | 2 | 0.8 | 2 | 0.6 |
| d2 (m) | 1 | 0.81 | 2 | 0.61 |
| d3 (m) | 2 | 0.75 | 1 | 0.75 |
| d4 (m) | 0.03 | 0.012 | 0.02 | 0.012 |
| d5 (m) | 0.6 | 0.8 | | |
| d6 (m) | 0.7 | 0.81 | | |
| d7 (m) | 0.7 | 1.25 | | |
| Optimum dimension of raft | 7.6 x 7.6 x 0.75 ($m^3$) | | 7.2 x 7.2 x 0.75 ($m^3$) | |
| of pedestal | 0.8 x 0.81 x 1.25 ($m^3$) | | | |
| Optimum cost | 78361 (Rs) | | 63640 (Rs) | |
| cost of steel | 9588 (Rs) | | 9727 (Rs) | |
| cost of conc. | 66507 (Rs) | | 51831 (Rs) | |

## 3.1 CONCLUSIONS

On the basis of obtained results and discussion presented earlier following conclusions can be drawn.

1. The developed procedure where in the optimal cost design of a rigid raft has been formulated as a nonlinear programming problem has been found to be quite efficient in isolating the optimal solution. The composite function developed by using extended penalty function approach proposed by Kavlie and Moe (1971) behaved very nicely during sequential minimization and converged to the optimal solution solution very fast. Adoption of Powell's Conjugate Direction method for multi-dimensional search and quadratic fit for linear search has been appropriate as evident from the fact that during the search for the minimum the minimization of the composite function led to the minimization of the objective function without any difficulty.

2. For saturated fine grained soils under undrained condition, the undrained shear strength plays a great role on the optimal dimensions of the raft when its value is less than a limiting value. However, above that value the effect of undrained shear strength is negligible. Thickness of the raft is not affected by the variation in the undrained shear strength.

3. For purely frictional soils there is no effect of $\phi$ ($20^o \leq \phi \leq 30^o$) on the optimum cost and optimum dimensions. In other words, for the range of parameters studied $\phi$ is an insensitive parameter within the range studied.

4. $C_c/(1+e_o)$ is a very sensitive parameter. Up to a certain value of the same the cost is not effected much. But, after that with the increase of the same cost, increases at a higher rate and beyond a certain value no solution can be obtained.

5. Modulus of elasticity $E_s$ has a great effect on optimum design of raft.

For very low value of $E_s$ it is not possible to make a raft foundation. For $c-\phi$ -soil when modulus of elasticity value is lower than a limiting value and Poisson's ratio equal to 0.499, solution cannot be obtained and at that value of $E_s$, cost of the raft is very high.

5. The design variable, $d_3$ ,i.e., depth of the raft is an insensitive variable with respect to all soil parameters taken into account in this study. Even if the load is increased final value of $d_3$ remains near about the minimum value that is imposed as a constraint. But with the increase of load other variable changes and base area of the raft increases.

6. In case of saturated fine grained soil, for the same column loading if the spacing between the two columns decreases then 'area ratio' ($A_r$) increases from the requirement of bearing capacity criteria. But, this not so prominent in case of general $c-\phi$ -soil. this is due to the fact that imposition of the geometric constraint overrides the design depth required from shear and moment consideration.

## 3.2 FUTURE SCOPE

On the basis of the present study one can do the following studies in future.

1. Optimum cost design of circular and rectangular/square raft treating these as plates on elastic foundation.

2. Optimum design of piled-raft system.

3. Optimum design of well foundation.

# Bibliography

[1] Bavikatti, S.S. and Hegde (1979), *"Optimum Design of Isolated Column Footing Using Sequential Linear Programming"*, International Conference on Computer Application in Civil Engineering, Roorkee.

[2] Bhattacharya, G. (1990), *"Seqential UnConstrained Minimization Technique in Slope Stability Analysis"*, Ph.D. Thesis., Dept. of Civil Engg., IIT Kanpur.

[3] Bowles, Joseph. E.(1982), *"Foundation Analysis & Design"*, McGraw-Hill Inc.,New York.

[4] Bowles, Joseph. E.(1988), *"Foundation Analysis & Design"*, McGraw-Hill Inc.,New York.

[5] Bromhead, E.N. (1986), *"The Slope Stability of Slopes"*, Survey University Press, New York.

[6] Brown,P.T. (1969), *"Numerical Analysis of uniformly loaded circular rafts on Elastic Layers of Finite Depth"*, Geotechnique,Vol. 19,pp. 301-306.

[7] Choudhury, R.N. (1978), *"Slope Analysis"*, Departments in Geotechnical Engineering 22, Elsevier Scientific Publishing Co., New York.

[8] Debeer,E.E. (1965), *"Bearing Capacity and Settlement of Shallow Footings on Sand"*, Symposium on Bearing Capacity and Settlement of Foundations,Duke Univ. pp.41-46.

[9] Desai, C.S. and Reese, C.C. (1970), *"Analysis of Circular Raft Footings on Layered Soils,"*, JSMFED, 90, SM4, pp 129-132.

[10] Fox, R.C. (1971), *"Optimization Method for Engineering Design"*, Addison Welesly, 2nd edition.

[11] Graham, J.(1984), *"Methods of Stability Analysis"*, Slope Instability, D.Brunsden and D.B.Prior, John Wiley and Sons Ltd..

[12] Hansen, J.B. (1970), *"A revised study of Extended Formula for Bearing Capacity"*, Danish Geotechnical Institute Bul. No. 28, Copengagen.

[13] IS: 456-1978., *"Code of Practice for Plain and Reinforced Concrete"*, Bureau of Indian Standards., New Delhi., 1981.

[14] IS: 1904-1986., *"Indian code of Practice for Structural Safety of Buildings: Foundations(second revision)"*, Indian Standards Institution, New Delhi.

[15] Kavlie, D. and Moe, J. (1971), *"Automated Design of Frame Structures"*, Journal of Structural Division, ASCE, 97, ST1, pp 33-61.

[16] Kishor, M. Parate. (1985), *"Settlement of a Two-Storey Framed Structure on Alluvial Soil"*, M.Tech. Thesis, Deptt. of Civil Engg., IIT Kanpur.

[17] Lambe,T.W. and Whitman,R.V.(1978), *"Soil Mechanics,SI Version,"*,John Wiley and Sons,New York.

[18] Madhan Mohan, R. (1988), *"Settlement Controlled Optimum Design of Shallow Footings"*, M.Tech. Thesis, Deptt. of Civil Engg., IIT Kapur.

[19] Mindlin,R.D. (1936), *"Force at a Point in the Interior of a Semi-infinite solid,"*, Physic. Vol.7.

[20] Mondal, S. A., (1988), *"Optimum Design of Shallow Circular Footings"*, M.Tech. Thesis, Deptt. of Civil Engg., IIT Kanpur.

[21] Murthy,V.N.S.(1992), *"Soil Mechanics and Foundation Engineering"*, Sai Kripa Technical Consultants,Bangalore.

[22] Mostyn, G.R. and Small, J. (1987), *"Method of Stability Analysis"*, Slope Instability and Stabilization, Walker and Fell (eds.), Balkena Rotterdam.

[23] Nash,D. (1987), *"A Compapative Review of Limit Equilibrium Methods of Slope Stability Analysis"*, Slope StabiEdited by Anderson, M.G. and Richards, K.S., John Wiley and Sons Ltd..

[24] Pandian,N.S.,Sridharan,A. and Sathidevi,U. (1994), *"Economical Considerations in the Design of Combine Footings"*, Indian Geotechnical Journal,Vol. 24,No. 4, pp. 378-409.

[25] Poulos,H.G. and Davis,E.H. (1974), *"Elastic Solutions for Soil and Rock Mechanics"*, John Wiley and Sons,Inc.,New York.

[26] Powell, M.J.D. (1964), *An Efficient Method for Finding Minimum of a Function of Several Variables Without Calculation Derivatives"*, Computer Journal, Vol. 7, No. 4, pp 303-307.

[27] Prakasa Rao,A.V.N.S.(1985), *"Effective Stress Interpretations of In-situ Test Data,"*, M.Tech. Thesis, Deptt. of Civil Engg.,IIT Kanpur.

[28] Rao, S S., (1984), *"Optimization Theory and Applicaions"*, Wiley Edition., New-Delhi.

[29] Schmertmann,J.H. (1970), *"Static Cone to Compute Static Settlement Over Sand,"*, A.S.C.E.,J.S.M.F.D.,Vol. 98,SM3, pp. 1011-1043.

[30] Sharma,K.G.,Nagpal,A.K. and Garg,M.K.(1984), *"Finite Element Analysis of Rafts Resting on Elastic Half-space"*, Indian Geotechnical Journal,Vol. 14,No. 1, pp. 28-39.

[31] Singh, A. (1987), *"Analysis of Circular Raft Foundation"*, International Book Traders, Delhi.

[32] Singh, V.K. (1982), *"Comparative Study of In-situ Tests"*, M. tech. thesis, IIT Kanpur.

[33] Skempton,A.W. and Bjerrum,L. (1957), *"A Contribution to the Settlement Analysis of Foundation on Clay"*, Geotechnoque, Vol. 7,No. 4,pp. 168-178.

[34] Subbarao, K.S., Narayanarao and Ranganathan, B.V. (1972), *"Plan Dimensioning of Footing Subjected to Uniaxial Moments"*, Indian Geotechnical Journal, January, Vol. 2, No. 1, pp 1-20.

[35] Kumar, Sushil., (1995), *"Structural Analysis and Design"*, Standard Publications, New Delhi.

[36] Teng,W.C.(1987), *"Foundation Design"*, Prentice-Hall of India Pvt. Ltd., New Delhi.